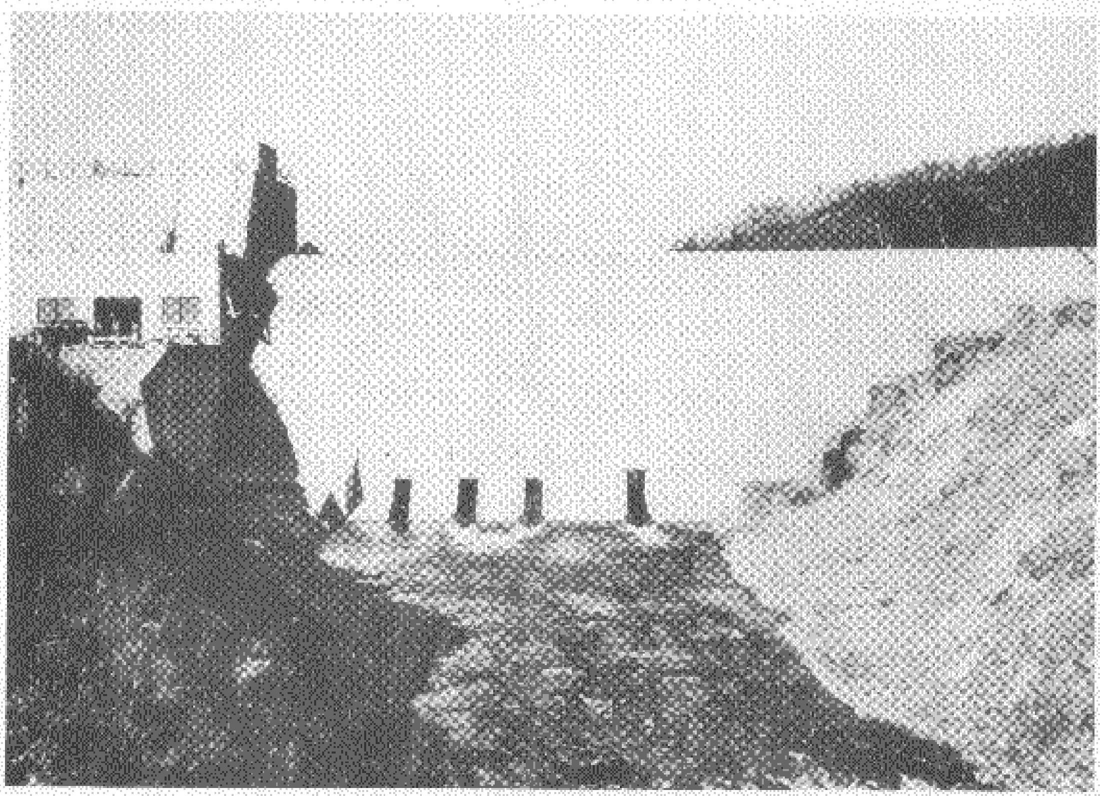
# BLACKWATER DAM



BLACKWATER RIVER - WEBSTER N. H.

*Constructed by*

CORPS OF ENGINEERS U. S. ARMY

## MERRIMACK VALLEY FLOOD CONTROL

# MERRIMACK VALLEY FLOOD CONTROL

# THE STORY

# OF

# BLACKWATER DAM

# BLACKWATER RIVER

CORPS OF ENGINEERS, U. S. ARMY

U. S. ENGINEER OFFICE BOSTON, MASS.

# BLACKWATER DAM AND RESERVOIR

## Pertinent Data

### Dam and Dikes

Type of structure....................Concrete and rolled earth fill

Maximum height of dam, feet (above deepest part of foundation)....70

Elevation of top of dam, above mean sea level..................584

Length of dam along axis, feet................................1,100

Aggregate length of dikes, feet...............................1,620

| | | |
|---|---|---|
| Volume of main dam, | earth fill, cubic yards................. | 152,000 |
| | rock fill, " " ................. | 42,000 |
| | concrete fill, " " ................. | 26,000 |
| | Total volume, cubic yards | 220,000 |

| | | |
|---|---|---|
| Volume of dikes, | earth fill, cubic yards.................... | 67,000 |
| | rock fill, " " .................... | 8,500 |
| | Total volume, cubic yards | 75,500 |

### Spillway and Outlets

Crest elevation, above mean sea level..........................566

Length, feet...................................................240

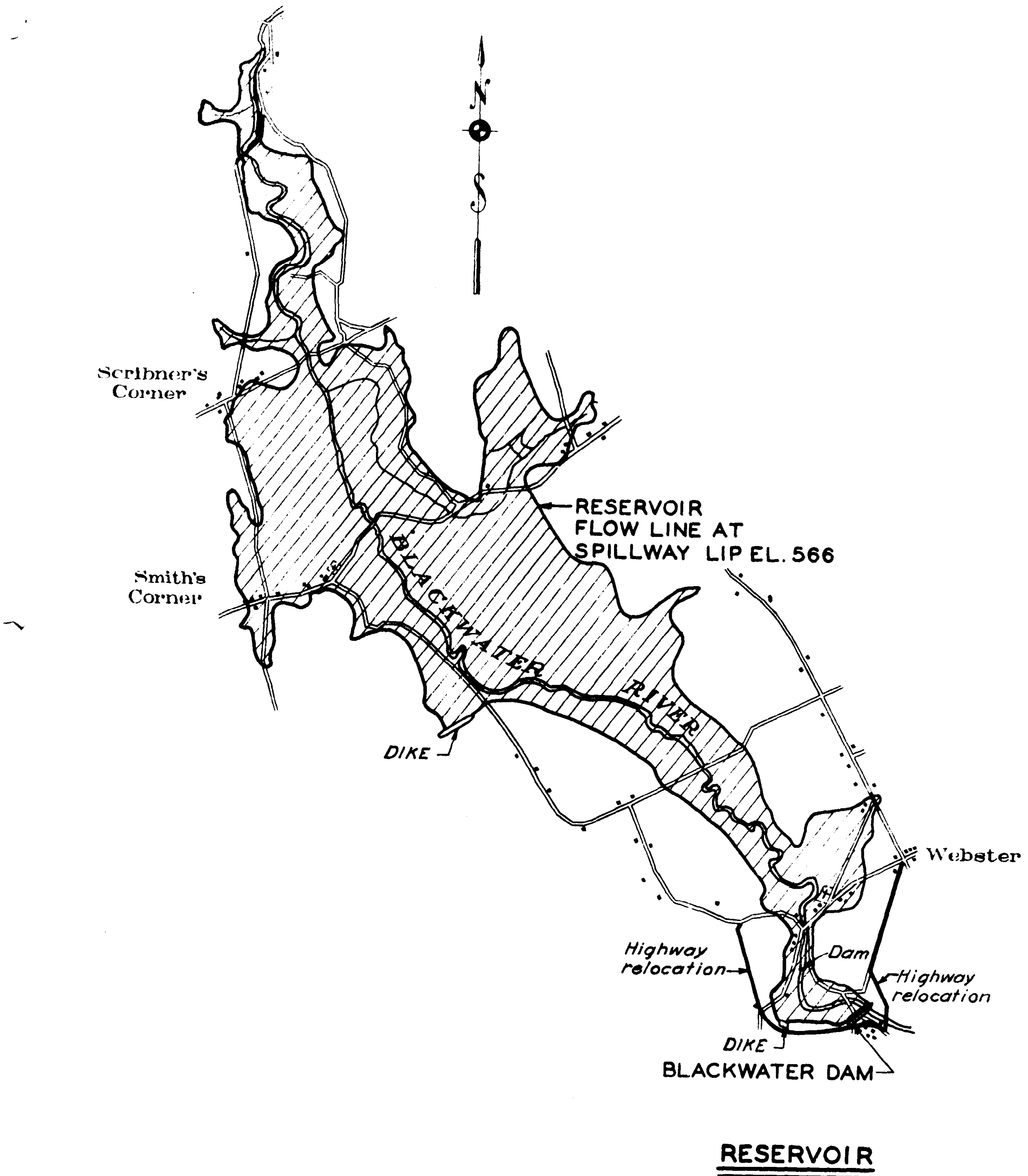
Size of outlets, 4 rectangular outlets - 1 ungated, 3'-6" x 6'-6"
- 3 gated, 3'-6" x 5'-3"

Elevation of invert of outlets.................................515

### Reservoir

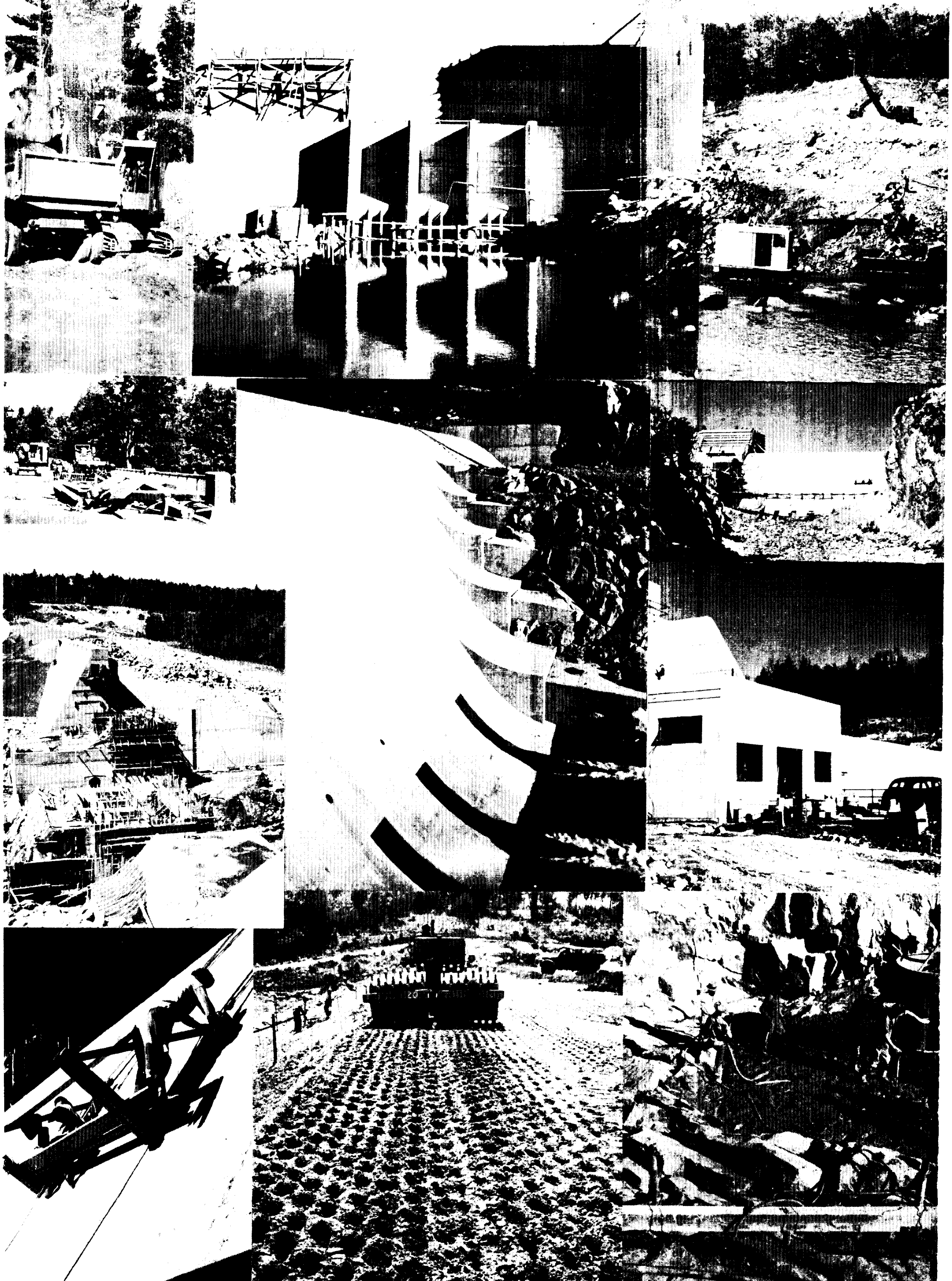
Drainage area, square miles.....................................127.5
Elevation of top of flood control pool..........................566.
Capacity in acre-feet...........................................46,000
Capacity in cubic feet................................2,004,000,000
Capacity in inches of run-off...................................6.8
Length of reservoir, pool full.......................about 7 miles
Area of reservoir, pool full.............................3,140 acres

Total estimated cost including construction, engineering, supervision, land and relocation of utilities....................................$1,300,000


Scribner's Corner
Smith's Corner
BLACKWATER RIVER
RESERVOIR FLOW LINE AT SPILLWAY LIP EL. 566
DIKE
Webster
Highway relocation
Dam
Highway relocation
DIKE
BLACKWATER DAM
N
S

## RESERVOIR

SCALE IN FEET
4000
0
4000
8000



DESCRIPTION OF PROJECT

During the history of New England and particularly in the last two decades, this section of the country has been afflicted with floods on the principal rivers which have resulted in the loss of life, destruction of public and private property and disrupted communication and transportation facilities. These floods reached a peak in destructive power in 1936 when record flood conditions prevailed on practically all of the large rivers of the northeast, paralyzing the economic life of the people over widespread areas. To provide protection from such disasters in the Merrimack River Basin, Congress authorized, by the Flood Control Acts of 1936 and 1938, the construction of a comprehensive system of reservoirs and other works. The State of New Hampshire in 1939 passed legislation enabling the Federal Government to acquire land for flood control purposes. The Blackwater Reservoir is part of the comprehensive plan for the Merrimack River Basin which, in conjunction with the Franklin Falls Dam now under construction, local protection works in operation or under construction, and other proposed reservoirs, will form a coordinated system for flood control affording the most economical protection for the Merrimack River Basin.

The Blackwater Reservoir, first to be completed under Federal Flood Control Acts in the New England States, is located on the Blackwater River in New Hampshire, 8.2 miles above the confluence with the Contoocook River, and 118.8 miles above the mouth of the Merrimack River of which the Contoocook is a principal tributary. The reservoir lies within the towns of Webster and Salisbury in the County of Merrimack, the Dam being situated just above the village of Swett's Mills in Webster, about 17 miles by highway northwest of Concord, New Hampshire.

The reservoir, which controls a drainage area of 127.5 square miles, has an area of 3,140 acres at the spillway lip (elevation 566.0) and a flood control storage capacity of 46,000 acre-feet, which is equivalent to 6.8 inches run-off over the drainage area. The river originates on comparatively steep mountain slopes with rapid run-off and forms at the confluence of three headwater tributaries about 20 miles by river above the dam site. The drainage basin is approximately 19 miles long with a maximum width of 12 miles with the greater part of the reservoir located in the main valley for about 7 miles between the Dam and West Salisbury. The reservoir area is composed of approximately 12% cultivated land, 9% pasture land, 74% wooded land and 5% water surface. Due to the sparse settlement and undeveloped character of the region, no relocation of railroads or villages was necessary. However, it has been necessary to relocate existing highways and cemeteries in the area. One highway relocation connecting Corser Hill in Webster with Swett's Mills and crossing the Blackwater River just below the dam site forms a link in the main route through this section. A second relocation, presently under construction, curves westward just below the earth embankment of the Dam. One cemetery within the reservoir area has been relocated and can be seen on the road connecting

Corser Hill and Dingits Corner. About 1,000 remains were involved in this relocation. The Government will purchase all the land lying in the reservoir basin up to the elevation of the spillway crest (elevation 566.).and the buildings on these lands have been sold to the highest bidder and are currently under process of demolition and removal.

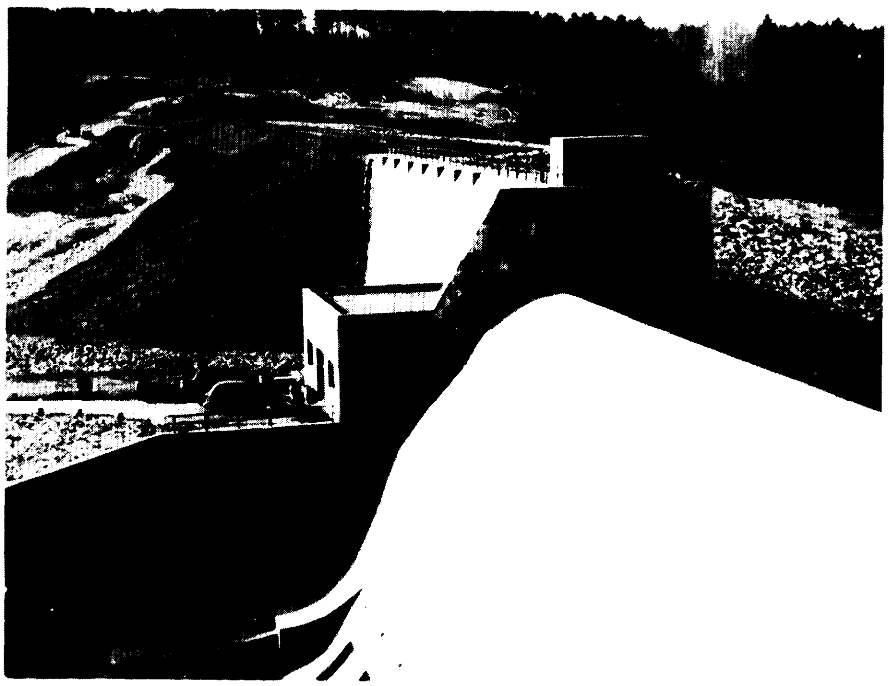
The Dam is located at a point where the river had incised a deep, narrow bedrock gorge. Bedrock on the site was either exposed or accessible over the entire dam site at comparatively shallow depths, rarely exceeding fifteen feet. The rock structure is generally a coarse granite and for the most part, especially in the river section, was much fractured but suitable for structure foundations.

The initial flood control development of the Blackwater Dam consists of a concrete spillway, a concrete non-overflow section, and an earth-fill retaining section. The concrete spillway is a wide-base, gravity-type Ogee section having a crest length of 240 feet and a maximum height of 70 feet above the deepest part of the foundation. The concrete non-overflow section is adjacent to the spillway and consists of a buttressed section 40 feet long, containing provisions for a future penstock intake and a wide-base gravity section 130 feet long connecting with the earth embankment. The embankment is a wide-base structure having a length of 700 feet and a maximum height of 60 feet. It is constructed of rolled earth-fill with a central impervious core and is protected by means of an upstream rock shell and a downstream rock-filled drainage toe. All masonry structures, the central core and downstream toe of the embankment are founded on bedrock. In addition to the main dam, one dike having a length of 1200 feet and a maximum height of 34 feet, and a second dike having a length of 420 feet and a maximum height of 20 feet have been constructed. These dikes are required in locations where the existing ground is at a lower elevation than the top of the dam and without which impounded water would escape from the reservoir.

The spillway consists of a mass concrete structure founded on bedrock with curved "buckets" to deflect the overflow parallel to the discharge channel bed. The gorge below the spillway has been enlarged to prevent possible overflow of the original narrow channel and will pass the designed spillway outflow without submerging the spillway crest. In the interior of the spillway structure, three chambers have been constructed, accessible by galleries from the outside, in which gate-operating mechanisms are located. The flood control outlets consist of one ungated rectangular opening 3' 6" x 6' 5" in cross-section and three gated outlets each 3' 6" x 5' 3", all located in the spillway section. The capacity of the outlets has been determined to provide for control of a flood derived from the maximum storms recorded in the northeastern parts of the United States. Flow through three conduits is controlled by means of slide gates. Each gate is embedded in the concrete as a unit with the gate leaf traveling vertically in guides. In each gated conduit there are two gates, one for ordinary

Above: Looking West from East abutment of dam at start of construction.

Below: Looking West from East abutment along downstream side of completed dam.



service, the second for emergency duty. The gates are operated by an oil-pressure system which develops a pressure of 500 pounds per square inch. On the upstream side of the spillway can be seen the entrances to these conduits with trash bars designed to stop all floating masses, such as tree stumps, which might otherwise plug the conduits and render them inoperative.

The equipment room houses a standby gasoline-electric generator unit for use in the event that the regular power supply becomes disrupted during flood periods. It will produce sufficient power to operate all the equipment installed at the dam. In this room are also the oil pumps, operated by electric motors, which build up the pressure necessary to raise and lower the gates. Two pumps have been provided, one for normal service and a second for emergency duty. A switchboard for the distribution of energy to the installed equipment is also located here. It will be noted that there are actually two complete installations for the operation of the dam from the source of power to the pumping unit and gates thereby eliminating the possibility that the gates might be inoperative during an emergency.

## CONSTRUCTION

The Dam and dikes were designed by the Corps of Engineers, U. S. Army, and constructed under a contract with A. S. Wikstrom of Bound Brook, New Jersey under the supervision of the District Engineer, Boston District, War Department. Construction of the Dam was started in April 1940 by stripping the dam site, which had previously been cleared by the Government. Excavation to bedrock along the center of the dam and rock excavation for the foundation of concrete structures were commenced. Along the centerline of the Dam holes were drilled alternately 20' and 40' deep which were grouted with Portland cement under pressure. This grouting program was for the purpose of sealing up any seams that existed in the rock foundation below the surface. Early investigations had shown that the natural rock in the dam area had been subjected to intense geologic activity and in order to prevent leakage through the Dam, these seams were sealed off with grout. Upon the completion of this work the borrow areas were opened and the placing of earth-fill commenced. The earth embankment was formed of a central impervious core material obtained about a half mile north of the dam site and of a pervious material taken from the large borrow area adjacent to the end of the embankment. These materials were placed in thin layers and tightly compacted with a sheepsfoot roller. In conjunction with this operation, the forming and placing of the concrete structures was started. It was necessary to construct a plant to process materials to be used for concrete aggregates as well as a batching plant where the processed material was mixed in the correct proportions and carried to the mixer. A borrow area containing gravel suitable for processing was located about three miles south of the Dam near Dingits Corner. In order to work on the spillway section it was necessary to divert the river, which was accomplished by means of a wooden flume and earth cofferdams which passed the low summer flows and enabled the foundation work to be carried on. At the same time construction of the dikes was started. The dikes are similar to the embankment section. At the close of the 1940 construction season most of the earth-fill sections were completed and the masonry structures commenced. The contractor stopped work in February and resumed in April. During the spring of 1941 the remaining spillway foundations were placed and the river was diverted through the conduits and the main earth embankment was completed. Rock excavation in the river gorge and the construction of the spillway and penstock intake structures continued throughout the summer. The building of the equipment room and the installation of machinery, piping and electrical system completed the construction program in November 1941.

## RESERVOIR OPERATION

No reservoir of water will exist behind the Dam in normal times as this area is reserved for storage in time of flood. Normal operation calls for the ungated and one gated outlet to be open and this will allow the passing of all ordinary river flow in an average year without forming any appreciable pool behind the dam. In time of flood the dam will act as a retarding basin, storing up water in the reservoir and at the same time discharging through the open conduits. Except in the case of extreme flood conditions, water will not store in sufficient volume to fill the reservoir and discharge over the spillway. The additional conduits are to be used principally for reducing the time necessary to empty the reservoir after the flood peak has passed the damage centers downstream. At Blackwater Dam it is anticipated that complete emptying of the reservoir from a full pool will be accomplished in nine days. As is evident, the action of these dams is one of retarding the flow in flood time, thereby reducing the peak and discharging the detained water later when the waters on uncontrolled streams have passed their peak discharge.

The Blackwater Reservoir will control 7% of the Contoocook watershed and approximately 2.6% of the entire Merrimack drainage basin. The annual flood control benefits are estimated at $140,000.

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This pamphlet distributed at the official opening of the reservoir and dam to public inspection on November 23, 1941.

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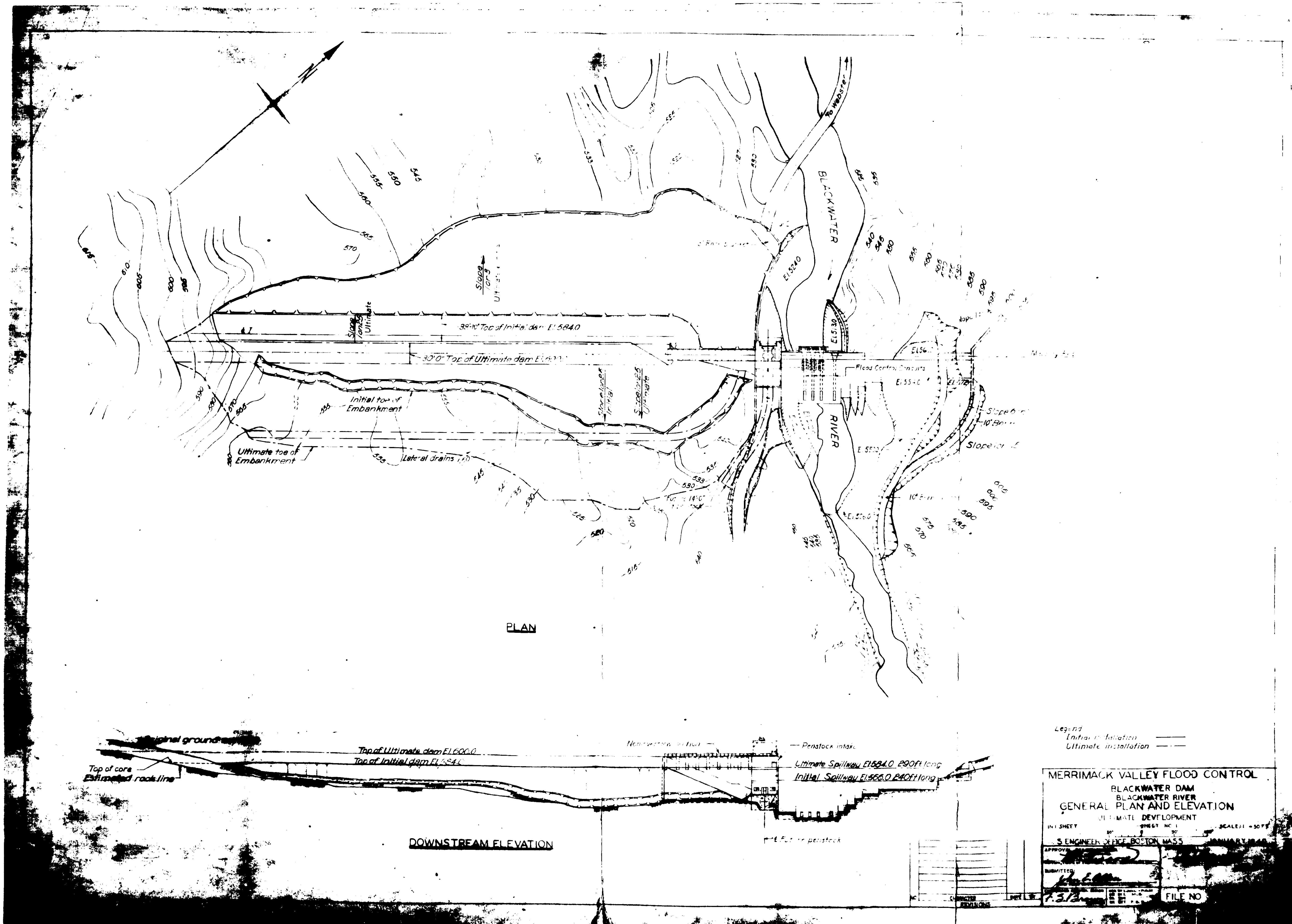
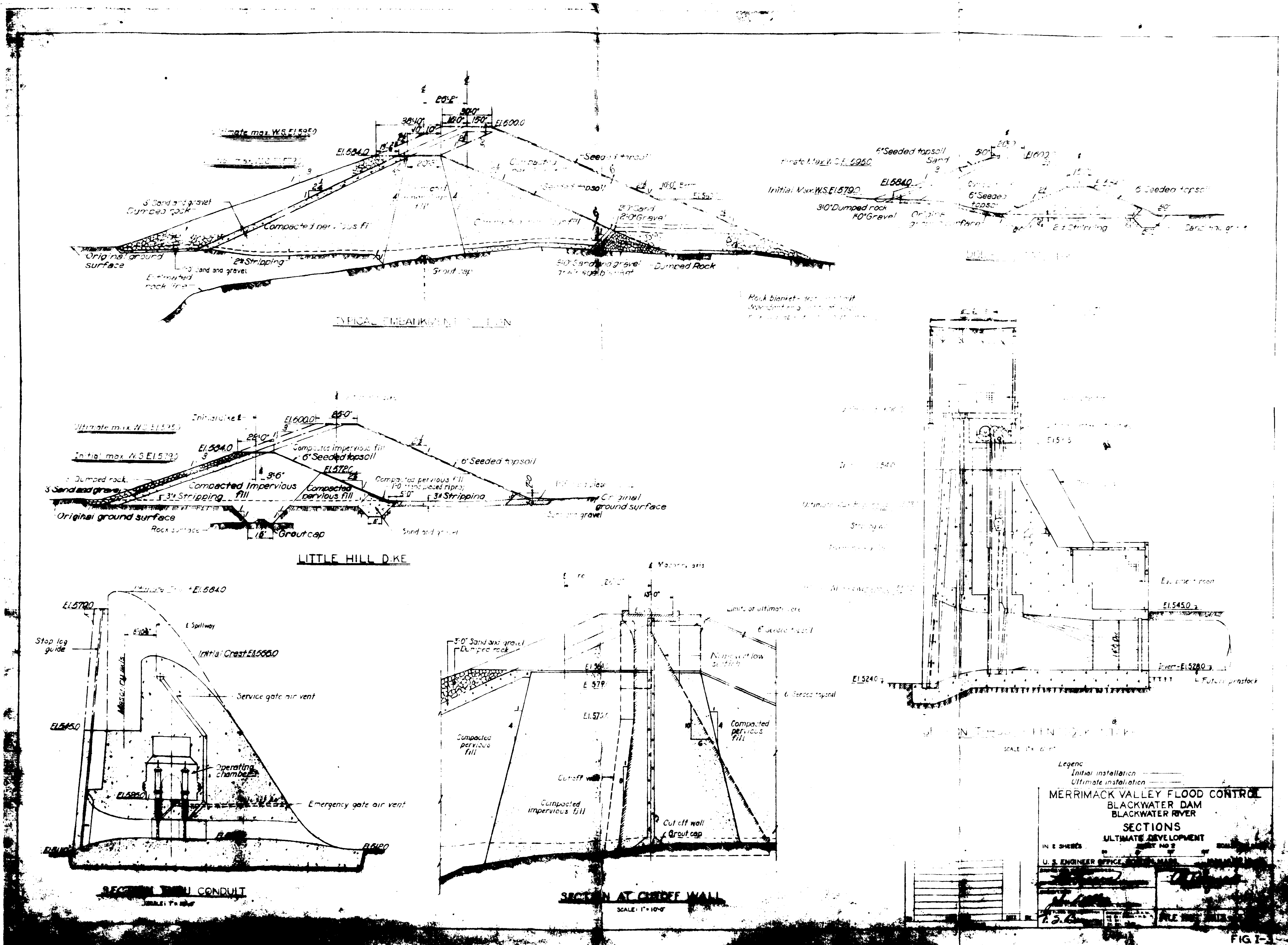


PLAN
DOWNSTREAM ELEVATION
Top of Ultimate dam El 600.0
Top of Initial dam El 584.0
Top of core
Estimated rock line
Penstock intake
Ultimate Spillway El 584.0 290 ft long
Initial Spillway El 566.0 240 ft long
Initial toe of Embankment
Ultimate toe of Embankment
Lateral drains
Flood Control Conduits
BLACKWATER
RIVER
To Webster
Legend
Initial installation
Ultimate installation
MERRIMACK VALLEY FLOOD CONTROL
BLACKWATER DAM
BLACKWATER RIVER
GENERAL PLAN AND ELEVATION
ULTIMATE DEVELOPMENT
FILE NO



Ultimate max. W.S. El. 595.0
El. 584.0
El. 600.0
Seeded topsoil
3' Sand and gravel
Dumped rock
Compacted pervious fill
Original ground surface
2' Stripping
Grout cap
Dumped Rock
LITTLE HILL DIKE
Ultimate max. W.S. El. 595.0
Initial max. W.S. El. 579.0
El. 584.0
El. 600.0
Compacted impervious fill
6" Seeded topsoil
Compacted Impervious fill
Compacted pervious fill
3' Stripping
Original ground surface
Rock surface
Grout cap
Sand and gravel
Initial Crest El. 566.0
Stop log guide
Service gate air vent
Operating chambers
Emergency gate air vent
El. 570.0
El. 545.0
El. 585.0
SECTION THRU CONDUIT
SECTION AT CUTOFF WALL
SCALE: 1" = 10'-0"
Compacted pervious fill
Compacted impervious fill
Cutoff wall
Cut off wall
Grout cap
6" Seeded topsoil
El. 545.0
El. 524.0
Legend
Initial installation
Ultimate installation
MERRIMACK VALLEY FLOOD CONTROL
BLACKWATER DAM
BLACKWATER RIVER
SECTIONS
ULTIMATE DEVELOPMENT

September 15, 1939

Revised December 16, 1939

# MERRIMACK VALLEY FLOOD CONTROL

## DEFINITE PROJECT REPORT

## FOR

## BLACKWATER RESERVOIR

Table of Contents

TEXT OF REPORT



APPENDIXES

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PLATES

WAR DEPARTMENT
UNITED STATES ENGINEER OFFICE
3RD FLOOR, PARK SQUARE BUILDING
BOSTON, MASS.

Sept. 15, 1939
Revised Dec. 16, 1939

DEFINITE PROJECT REPORT FOR BLACKWATER RESERVOIR -
MERRIMACK RIVER BASIN FLOOD CONTROL

1. Project Authority. The Flood Control Act approved June 28, 1938, provides that, "The general comprehensive plan for flood control and other purposes (for the Merrimack River Basin), as approved by the Chief of Engineers pursuant to preliminary examinations and surveys authorized by the Act of June 22, 1936, is approved and the project for flood control in the Merrimack River Basin, as authorized by the Flood Control Act approved June 22, 1936, is modified to provide, in addition to the construction of a system of flood control reservoirs, related flood control works which may be found justified by the Chief of Engineers."

2. Previous Investigations. The results of an investigation of a system of flood control reservoirs and related flood control works for the Merrimack River Basin are contained in a preliminary examination and survey report authorized by the Flood Control Act approved June 22, 1936, and published as House Document, Public No. 689, 75th Congress, 3d Session. This report proposes several alternate combinations of reservoirs to serve as a general comprehensive plan of flood control in the Merrimack Basin, in all of which the Franklin Falls Reservoir now authorized for construction forms a component part. Other reservoirs proposed for selection as parts of these systems are located mainly in the Contoocook Valley. The Riverhill Reservoir, the largest of this group, was recommended for first priority to supplement Franklin Falls and has been selected by the Chief of Engineers as a definite project.

3. Practical objections have developed to the further prosecution of the Riverhill project. As explained in letter from this office to the Chief of Engineers, subject: "Selection of Definite Project - Merrimack River Flood Control", dated August 25, 1939, a further study is being made with a view to replacing Riverhill with an equivalent system of reservoirs in the Contoocook Valley. This study has progressed to the extent of positively indicating that any system of replacement should include the Blackwater Reservoir. The investigations have been reduced virtually to the consideration of two plans, a combination of Blackwater and the Contoocook River Flood Diversion Project, for which a favorable examination has been completed and was pre-

sented with letter to the Chief of Engineers cited above, and a second plan now being studied which was suggested by the Federal Power Commission. This latter plan comprises a system of ten small reservoirs in the Contoocook Valley, of which the Blackwater project would form a principal unit regardless of any modification which may be found desirable in the present study of this plan.

4. Location and Description of Area. Blackwater Reservoir is located on the Blackwater River in New Hampshire, 8.2 miles above the confluence with the Contoocook River and 118.8 miles above the mouth of the Merrimack River, of which the Contoocook is a tributary. The reservoir lies within the towns of Webster and Salisbury in the county of Merrimack, the dam being situated just above the village of Swetts Mills in Webster, 12.5 miles by highway northwest of Concord, N. H. The reservoir, which controls a drainage area of 127.5 square miles, has an area at spillway lip (El. 566) of 3,140 acres and a flood control storage capacity of 46,000 acre-feet, which is equivalent to 6.8 inches of run-off. The reservoir area is composed of approximately 12 per cent cultivated land, 9 per cent pasture land, 74 per cent wooded land, and 5 per cent water surface. No railroads or villages are involved and settlement and development in the region are sparse.

5. The Blackwater Reservoir, in conjunction with the Franklin Falls Reservoir now under construction and the proposed Contoocook River Flood Diversion Project, for which a preliminary definite project report was submitted to the Chief of Engineers with letter dated September 16, 1939, will form a coordinated system of flood control reservoirs in the Merrimack Basin approaching in both cost and predicted benefits the presently selected Franklin Falls and Riverhill Reservoirs.

6. Definite Project Plan. In view of the value of the Blackwater site for future combined storage and power development (see Appendix E), it is proposed to construct the Blackwater Dam initially for flood control, with provisions to facilitate future alteration for a multi-purpose development. The estimated additional cost of these provisions is $206,000.

7. Description of Structures. The proposed initial flood control development of the Blackwater Dam consists of a concrete spillway, a concrete non-overflow section, and an earth fill retaining section (see Plates VI, VII, and VIII). The concrete spillway is a wide-base, gravity-type Ogee section having a crest length of 185 feet and a maximum height of 56 feet. The concrete non-overflow section is adjacent to the spillway and consists of a buttressed section 60 feet long containing provisions for a future penstock intake and for future flood control outlet gates and a wide-base gravity section 140 feet long connecting with the earth embankment. The embankment is a semi-wide-base structure having a length of 770 feet and a maximum height of 60 feet. It is constructed of rolled earth fill with a central impervious core and is protected by an upstream rock shell and downstream rock-filled drainage toe. All

masonry structures and the impervious core and drainage toe of the embankment are placed on bedrock, which generally is available over the dam site within depths of 0 to 15 feet from the present ground surface. In addition to the main dam, one dike having a length of 1,200 feet and a maximum height of 34 feet is required for the flood control reservoir.

8. Hydrology and Hydraulics.- Operating under the maximum design surcharge of 13 feet, the spillway will pass a flow of 42,800 second-feet, which is the outflow derived from routing the spillway design flood through the reservoir. The peak inflow of this flood is 66,500 second-feet, which is equivalent to a run-off of 520 c.f.s. per square mile over the 127.5 square mile tributary drainage area and which gives a coefficient of 5,900 for "C" in the formula $Q = C\sqrt{A}$.

9. The initial flood control outlets consist of one ungated rectangular opening, 3'-6" x 6'-5" in cross-section, and three gated outlets, each 3'-6" x 5'-3", all located in the concrete spillway section. The ungated outlet and one gated outlet fully opened will be used initially to obtain a maximum design discharge of 2,000 second-feet at El. 566. The additional gated outlets will be available for emptying the reservoir, bringing the total capacity to 3,760 second-feet. The capacity of the reservoir to spillway crest and the capacity of the outlets have been selected to provide control of a reservoir design flood derived from the maximum storm recorded in the New England area. The outlets will discharge into the natural rock channel of the river in remote location with respect to the earth embankment. A more complete description of the hydrologic and hydraulic design of this project, with supporting data, is contained in appendixes A and B herewith.

10. Cost Estimates.- The estimated first cost of this project for flood control, with provision for future conservation and power development, is summarized below. Detailed cost estimates are contained in appendix F.

| | |
|---|---|
| Land and Relocation Costs | |
| Land, buildings, and rights-of-way . . . . . | $ 241,000 |
| Highway relocation . . . . . . . . . . . . . . | 130,300 |
| Cemetery relocation . . . . . . . . . . . . . | 64,800 |
| Utility relocation . . . . . . . . . . . . . . | 20,300 |
| Construction Costs | |
| Dam, dikes, and reservoir clearing . . . . . | 818,600 |
| Total Estimated Cost . . . . . . . . . . | $1,275,000 |

11. Economic Study.- The results of an economic study of this project are contained in the tabulation following.

ECONOMIC ANALYSIS - BLACKWATER RESERVOIR

| | First Cost | Annual Cost | Annual Benefits: Flood Control | Annual Benefits: Power | Annual Benefits: Total | Ratio, Annual Benefits to Costs |
|---|---|---|---|---|---|---|
| Blackwater* | $ 1,275,000 | 62,700***<br>45,700**** | $140,000 | $ -- | $ 140,000 | 2.23***<br>3.06**** |
| Blackwater** | 3,730,000 | 207,300 | 140,000 | 250,500 | 390,500 | 1.89 |
| Blackwater* and Franklin Falls | 8,775,000 | 413,600***<br>396,600**** | 763,700 | -- | 763,700 | 1.85***<br>1.93**** |
| Blackwater** and Franklin Falls | 11,230,000 | 558,200 | 763,700 | 250,500 | 1,014,200 | 1.82 |
| Blackwater*, Franklin Falls, and Contoocook River Flood Diversion Project | 19,101,000 | 890,300***<br>873,300**** | 907,500 | -- | 907,500 | 1.02***<br>1.04**** |
| Blackwater**, Franklin Falls, and Contoocook River Flood Diversion Project | 21,556,000 | 1,034,900 | 907,500 | 250,500 | 1,158,000 | 1.12 |

* - Flood control project with dam, designed to facilitate future raising.
** - Multiple-purpose project for flood control, conservation, and power.
*** - Annual charges for Blackwater are based on $1,275,000 (first cost of project with provision for future raising).
**** - Annual charges for Blackwater are based on $965,000 (future flood control share of combined development cost prorated on basis of storage. See Appendix E).

12. Local Cooperation. No local cooperation is required for this project as authorized by the Flood Control Act of 1936 amended June 28, 1938. Consent by the State of New Hampshire to the construction of the Blackwater Reservoir is provided in "An act consenting to the acquisition of lands by the United States for flood control and navigation purposes" approved May 31, 1939.

13. Time Required for Construction. Provided operations are begun early in the construction season, about April 15th, the earth fill of the embankment can be completed by November 1 of the same calendar year and all items of work can be completed by the end of that calendar year.

14. Results Expected. The Blackwater Reservoir will control 17 per cent of the Contoocook watershed and approximately 2.6 per cent of the entire Merrimack drainage basin. The annual flood control benefits are estimated at $140,000, and stage reductions for a flood similar to that of March 1936 at Manchester, New Hampshire, will be about 1.1 feet. The Blackwater Reservoir, in conjunction with the Franklin Falls Reservoir and the Contoocook River Flood Diversion Project, will control 32.4 per cent of the total Merrimack Basin area and will afford benefits of ~~$907,500~~ annually.

15. Recommendations. It is recommended that the Blackwater Reservoir be approved for immediate prosecution and that provisions be made in the initial dam to facilitate future alteration to include conservation storage and power development.

DEFINITE PROJECT REPORT FOR BLACKWATER RESERVOIR -
MERRIMACK RIVER BASIN FLOOD CONTROL

## APPENDIX A

## HYDROLOGY OF THE BLACKWATER RIVER DRAINAGE BASIN

### Part I - The Spillway Design Flood

1. Reference.- Reference is made to Engineer Bulletin, R.& H. No. 9, 1938, subject, "Spillway Capacities," which directs, in paragraph 19, that a hydrology report will be submitted to the Office of the Chief of Engineers for approval as the first step in the final design of a project.

2. Project Description.- The Blackwater dam site is located on the Blackwater River, a tributary of the Contoocook River. The dam is approximately 8-1/2 miles above the mouth of the river in the village of Swetts Mills in Webster township, N.H. The reservoir controls a drainage area of 127.5 square miles, has an area at El. 566 of 3,140 acres, and a storage capacity of 46,000 acre-feet, equivalent to 6.8 inches run-off over the drainage area. The initial project, for flood control only, proposes use of the reservoir as a retarding basin with fixed outlet capacity, consisting of one gated and one ungated outlet, total capacity 2,000 second-feet at El. 566. Two additional gated outlets are available if desired to reduce the time of emptying the reservoir. The concrete overflow spillway is 240 feet long, crest at El. 566, and maximum height of 65 feet.

3. Basin Characteristics.- The drainage basin of the Blackwater River above the dam site is approximately 19 miles long, with a maximum width of about 12 miles. The river originates on comparatively steep mountain slopes with rapid run-off. The three headwater tributaries, comprising about one-half the total drainage area, combine in the two-mile reach above Potter Place, about 20 miles by river above the dam site. The total fall in the reach to the dam site is 100 feet, 40 feet of which are in the two-mile reach near West Salisbury; hence in the remaining 18 river miles the drop is about 60 feet, or about 3 feet per mile. The main valley is about 15 miles long and provides considerable valley storage, particularly in the reach below West Salisbury, that is reflected in the distribution values. Lakes, ponds, and swamps comprise about 8 square miles, or over 6% of the drainage area.

4. Stream Flow Data.- A United States Geological Survey stream gaging station is located on the Blackwater River near Webster about two miles downstream from the dam site and has been

in operation since October 1934. It measures the flow from a drainage area of 129 square miles, only 1.5 square miles more than the drainage area at the proposed dam site. Records at this station have been used as the basis for stream flow data and studies of the distribution values. Staff gage readings and discharge measurements have been taken by this office at the Swetts Mills dam site to establish a tailwater rating curve and correlated to discharges recorded at the U. S. Geological Survey station. Discharge records based on one or two daily gage readings have been obtained by the U. S. Geological Survey further downstream near Contoocook (drainage area 134 square miles) from 1918-1920 and 1927-1934.

5. <u>Precipitation Stations and Records</u>.- The only precipitation station in the Blackwater Basin is a non-recording station established by this office at South Danbury in January 1937. The records obtained at this station are considered exceptionally good for this type of station. The U. S. Weather Bureau has maintained for many years stations adjacent to the watershed. Stations used for this study are shown on plate 13 and on plate 14. The only recording rain gage applicable to the Blackwater River Basin is located at Concord, N.H., about 12 miles distance from the dam site. The mass curves of precipitation for the non-recording gages were drawn with and conforming to the Concord recording rainfall data.

6. <u>Distribution Graph</u>.- The distribution values for the inflow hydrograph for Blackwater Reservoir were based on an analysis of the September 1938 flood. This flood had a peak value of ~~7,200~~ c.f.s. and is by far the maximum flood of record uninfluenced by snow run-off. The March 1936 flood, with a peak of 11,000 c.f.s., is the maximum of record but was primarily due to heavy snow melt; hence, because of uncertainties in required assumptions to utilize this flood for distribution values, it was not considered suitable for a check on the 1938 flood distribution values. A search for floods recorded at the present U. S. Geological Survey gaging station not affected by snow run-off gave 9 minor rises, with peak values less than 800 c.f.s. At the former U.S. Geological Survey station, with one or two daily readings, the maximum recorded discharge was 2,250 c.f.s., with possible backwater effect from the Contoocook River. A preliminary study of these minor floods showed peak distribution values appreciably lower than those from the 1938 flood due to the large effect of valley storage and regulation. The results could not be considered a check. The hydrograph recorded at the U.S. Geological Survey gage can be considered an outflow from the reservoir area without appreciable correction. In order to obtain an inflow hydrograph, this flood hydrograph was routed backwards through the valley storage of the reservoir site. The hydrographs are shown on plate 15.

7. Plate 14 gives the mass curves of precipitation for the stations in and adjacent to the Blackwater Basin for the flood of September 1938. Six-hour rainfall values were obtained from these curves and distributed over the basin using Thiessen's weighting. The total weighted rainfall for the storm is 7.81 inches in 72 hours. The flood hydrograph, after deducting base flow, showed a run-off of 4.64 inches. Thus, the total infiltration and other losses amounted to 3.17 inches. The average rate of these losses was computed, based on weighted hourly rainfall and total losses, to be 0.087 inch per hour. The rainfall and run-off are shown in bi-hourly values on plate 15. Six-hour distribution values were obtained for this inflow hydrograph, using six-hour rainfall values. The procedure and values used are shown on plate 16. The reconstructed hydrograph, using the derived distribution values, shows a satisfactory check with the original hydrograph. The unit hydrograph, based on one inch of run-off occurring in a period of six hours, is shown in plate 17. When this unit graph was used to derive the computed spillway flood, the resulting peak inflow gave a Myer rating of only 2830. Because of the comparatively low peak of the flood used in deriving the distribution values and the uncertainties of the effect of valley storage on the values, it was decided to increase the unit graph peak so that the resulting computed spillway flood would have a peak inflow with a Myer rating of at least 4000. The time of lag was reduced to maintain a reasonable relation to the higher peaked distribution values. It was found that reducing the time of lag tended to decrease the surcharge resulting from a flood of given volume and peak. Therefore, a time of lag of 16 hours was selected rather than a lesser value. The revised unit graph adopted is shown as a solid line on plate 17.

8. The following terms are used, as defined in Engineer Bulletin, R. & H. No. 9, 1938: (a) Reservoir Design Flood; (b) Computed Spillway Flood; (c) Spillway Design Flood; (d) Freeboard; (e) Freeboard Storage; (f) Surcharge; (g) Surcharge Storage; (h) Wave Height; (i) Wind Setup.

9. Maximum Storm.- The rainfall data for determining the maximum storm has been obtained from values furnished the Providence Office by the hydro-meteorological section of the U. S. Weather Bureau. The data are shown on plate 18. Instead of the value shown for 127.5 sq. mi. area on the six-hour curve on plate 18, a higher value plotted as a point on plate 18, was used upon recommendation of the hydro-meteorological section of the Office, Chief of Engineers. Comparison was made of the 24-hour and 48-hour depth-area curves from this storm with those for the total September 1932 and October 1903 storms and the 48-hour envelope curves developed by the Pittsburgh District Office, and were found to give materially higher values of rainfall for a 127.5 sq. mi. drainage area. Detailed study for this size drainage area of maximum storm data based on actual maximum storms of record (com-

parable to that obtained for the Franklin Falls project) is not available; hence, no absolute comparison can be made of the relative flood magnitude of summer and of spring storms with snow run-off. However, it is believed that for the small drainage area of the Blackwater River the much higher intensity of summer storms will prove to give appreciably greater peak values.

10. Computed Spillway Flood.- The computed spillway flood was based on the derived distribution values and on the rainfall curves shown on plate 18 and is shown on plates 19 and 20. This is a summer storm and consequently an assumed minimum infiltration rate of 0.083 inch per hour, or 0.5 inch per six-hour period was adopted. This is less than the average rate of 0.087 inch per hour determined from the September 1938 flood, where conditions of excessive rainfall during July and the three weeks in September prior to the flood were productive of minimum infiltration rates. A base flow of 2 c.f.s. per square mile was added to the flow run-off. Data pertaining to this flood are briefly summarized as follows:

| | | |
|---|---|---|
| Rainfall | - | 19.4 inches in 48 hours |
| Run-off | - | 15.8 inches |
| Volume | - | 111,700 acre-feet (including base flow) |
| Peak inflow | - | 49,300 c.f.s. |
| Peak outflow | - | 32,200 c.f.s. |
| Maximum pool elevation | - | 576.8 |

## Determination of Spillway Design Flood

11. Reservoir Operation Assumptions.- It is assumed that the reservoir will be filled to normal maximum pool elevation of 566 feet M.S.L., which is the crest of spillway, at the beginning of the spillway design flood. The outlets are assumed to be inoperative.

12. Spillway Rating Curve.- The spillway rating curve (plate 21) was computed by the weir formula $Q = CLH^{3/2}$, where values of "C" for an unsubmerged Ogee section were taken from 3.0 to 3.8 at maximum head.

13. Method of Routing.- As the computed spillway flood was based on distribution values for inflow to the reservoir, the flood was routed through the reservoir using the gross storage. (Reference is made to appendix F of the "Survey Report for Flood Control, Merrimack River," dated April 1, 1938, for a detailed discussion of the reservoir routing used by the Boston Office.)

14. Spillway Design Flood.- The computed spillway flood was routed through the reservoir to obtain the reservoir discharge and the maximum water surface elevation. The inflow ordinates of the computed spillway flood were then increased 50 and 100 per cent and similarly routed through the reservoir. The data derived from these three routings are shown graphically on plate 22, where the per cent of computed spillway flood is plotted, (a) against the pool elevation in feet above mean sea level, and (b) against the peak inflows and outflows. After carefully considering (see following paragraph) all the factors itemized in the Engineer Bulletin, R. & H. No. 9, 1938, subject: "Spillway Capacities," paragraph 11, it was decided to adopt, as the "Spillway Design Flood" for the Blackwater Dam, the computed spillway flood plus a factor of safety of 35 per cent. The resulting spillway design flood is shown on plate 23.

15. Discussion of Factor of Safety.- In selecting 35 per cent as the factor of safety to be applied to the computed spillway flood, the following items, as enumerated in the Engineer Bulletin referred to in preceding paragraph, were considered:

a. The long-time rainfall records are too few to cover adequately all large storms of record in this district. Results of the storm studies now being made by this office, and other data as outlined in paragraph 9 above, show that the spillway design storm used gives rainfall values materially in excess of the maximum storms experienced. The design storm, based on data available in this office, is believed to give maximum rainfall values.

b. The maximum rainfall values used as shown on plate 18 for 127.5 square miles are about twice those for a like area in the September 1932 storm and exceed by a greater proportion the rainfall values for smaller drainage areas. The maximum point rainfall values available in this office were obtained from unofficial records. These values are 10 inches in about two hours at Peterboro, N.H., during the September 1938 storm and 11 inches in about three hours obtained during a cloudburst August 1939 at Baldwin, Maine. These values are over 50 per cent less than those shown for 3-hour rainfall on a one-square-mile drainage area. Hence, rainfall values given on plate 18 are greater than those recorded in information available in this office. There appears to be little possibility that they will be exceeded.

c. The flood of September 1938 makes available an accurate determination of discharge quantities and reasonably accurate rainfall data. This is the only summer flood suitable for study, and with the excessive rainfall prior to the flood, should give values of infiltration closely approaching the minimum. The analysis of the September 1938 flood (plate 15) indicated a rate of infiltration of 0.087 inch per hour. A lower value of 0.5 inch per 6-hour period (0.083 inch per hour) was used in the computed spillway flood. Expressed in terms of per cent run-off, this is equivalent to 88 per cent run-off for the maximum 24-hour precipitation.

d. The unit hydrograph (plate 17) and distribution graph (plate 15) are based on an analysis of the September 1938 flood, the second largest flood of record. Study of the values obtained indicated the desirability of peaking the values as described in paragraph ~~5~~7. The computed spillway flood peak value is approximately 7 times that of the 1938 flood. However, the surcharge storage of 47,000 acre-feet available for the spillway design flood, with total volume of 150,000 acre-feet, is believed to minimize the inaccuracies inherent in the determination of the distribution values and the extrapolation of these values to a flood with peak value about 9-1/2 times that of the 1938 flood.

e. Errors in routing the design flood through the reservoir because of inaccuracies of assumed storage volume, backwater curves, and time of water travel are believed to be negligible. As the assumption is made that the reservoir is full at the beginning of the flood, the time of water travel is practically instantaneous through the reservoir. The spillway flood is computed as an inflow hydrograph and consequently consideration of the effect of valley storage in the reservoir was not required. The capacity curve of the Blackwater Reservoir is based on a topographic survey of the reservoir from which the values used for surcharge storage are obtained. There should be no appreciable error in the storage capacity curves or in the resulting reduction to the inflow peak obtained by routing the spillway flood through the surcharge storage.

f. Whether or not the outlet conduits are gated does not affect the criterion used, namely that during the duration of the spillway flood the outlets are assumed inoperative. Should the outlets actually be used for discharges during the spillway flood, the peak surcharge would be only slightly reduced. There is the possibility that the reservoir may be developed for power in the future. If this is done, the present proposal is to raise the crest of spillway to El. 584 and lengthen the spillway in order to reduce the surcharge. This will not change the basic spillway design storm or conditions, except to provide additional surcharge storage for a given increment of spillway surcharge.

g. The spillway is a straight overflow Ogee section. The spillway discharge rating has been computed with conservative values of "C" and it is believed that the spillway discharge capacity will actually exceed the computed values.

h. The freeboard storage in the Blackwater Reservoir is a considerable item. Between El. 579 and top of dam at El. 584 there are 22,000 acre-feet of storage, equivalent to 3.2 inches. There is a definite factor of safety against overtopping of the dam that may be attributed to this freeboard storage and spillway capacity, requiring a flood over twice the computed spillway flood to overtop the dam.

16. Summary.- In view of the items discussed in the preceding paragraphs, it is believed that a 35 per cent factor of safety applied to the computed spillway flood is adequate. The hydrograph of this flood is shown on plate 23 with the outflow-stage hydrographs and reservoir elevations. Data concerning this flood are summarized as follows:

Selected Spillway Design Flood
(Computed Spillway Flood Plus 35 Per Cent)

148,300
Volume = ~~149,700~~ acre-feet (including base flow)

= 22.0 inches of run-off from a 48-hour period of rainfall

Peak inflow = 66,500 c.f.s.

Peak outflow = 42,800 c.f.s.

Maximum pool elevation 579.0 ft. above M.S.L.

Freeboard 5.0 ft.

## Part II - Reservoir Design Flood

17. Maximum reservoir capacity is governed by the physical limitations of the dam site and by the size of the maximum probable flood. The former is determinable from the field surveys, the latter is not definitely determinable because of limited length of adequate discharge records. Another factor in the physical limitation to storage is the outlet discharge, governed by channel capacity and related to the storage proposed and reservoir design storm used. It is desirable to discharge from the outlets a flow not exceeding channel capacity or limiting the flow to that value which will not cause flooding on the tributary. This item of channel capacity is not important on the Blackwater River from the standpoint of flood damage in the reach from the dam to the Contoocook River. The greater the storage provided for the given storm, the smaller the outlet discharge and the greater the flood reduction obtained downstream. However, there is a practical limitation to the reduction of outlet discharge in that a flood control reservoir should be emptied in a reasonable time to provide for the possible condition illustrated by the 1936 flood where two major storms produced peak floods within a week of each other.

18. Related to all of the above factors is the economic consideration limiting the cost of the storage capacity to that which will give the greatest ratio of benefits to costs.

19. The reservoir design flood formerly used was based on economic consideration wherein it appeared desirable to limit the size of the reservoir to control a flood produced by a rainfall of about 100-year frequency, and also to empty the reservoir in a reasonable length of time. To arrive at such a storm, long-time rainfall records at Concord were studied. It was found from these records that a 100-year rainfall of 3-days' duration amounted to about 8 inches. With this storm a high run-off factor of 90 per cent was applied. In order to justify this high run-off factor, a 3-day storm of 1-1/2 inches of rainfall per day was added prior to the 100-year 3-day storm. This gave a flood volume that, in its effect on reservoir capacities, was about equivalent to the 1936 flood, the maximum flood of record. The flood obtained from these assumptions, however, must be considered as one of less frequent occurrence than once in 100 years because of the assumptions made of the 3-day prior rainfall and the high run-off coefficient. This flood, when routed through the reservoir with an outlet discharge of about 1,800 c.f.s., gave a spillway lip or maximum pool elevation of 566 feet.

20. An approach to the maximum probable flood may be had by consideration of the maximum available storm of record. Recent study of recorded maximum storms show that apparently the greatest storm of record in New England was that of September 1932, with principal center at Westerly, Rhode Island. This storm compares in magnitude with others outside of New England that are considered as possible of occurrence in New England. This office has detailed information on the 1932 storm, and it is considered as a rational basis for computation of a reservoir design flood approaching the maximum probable flood. Utilizing this flood with the most severe condition of run-off, comparable with that obtained in the 1938 flood, gives a severe criterion for selection of reservoir capacity. Plate 28 shows the depth-duration curves of the 1932 storm used for deriving the values of rainfall, while plate 29 shows the constructed inflow hydrograph and the resulting outflow-stage hydrographs using the revised maximum outlet discharge of 2,000 second-feet at El. 566.

21. This storm produces an extremely high peak with moderate volume. When routed through the reservoir, the assigned storage capacity was found to be sufficient to control the flood, except for a moderate spillway outflow of 1,600 c.f.s. The rare frequency when this small flow in excess of the conduit capacity will occur is considered a negligible factor. Hence, it is believed that the flood based on the 1932 storm is a reasonable criterion for a reservoir design flood based on summer rainfall conditions.

22. The March 1936 flood, in most cases on the Merrimack Basin, exceeded in peak value, and definitely in volume, any flood of record. Since, in reservoir flood design, volume is of primary importance, reservoir design in the Merrimack Basin must recognize the spring flood as the possible governing criterion. Storage provided in the Blackwater Reservoir is found to be ample to control the 1936 flood with the proposed outlet capacity. (See plate 24.)

23. Study of all the factors involved in consideration of the control of the Blackwater River floods led to the choice of 46,000 acre-feet of flood control storage. The possibility that there may be some excess flood control storage in the Blackwater Reservoir should be considered a desirable condition in connection with the comprehensive scheme of flood control on the Merrimack to increase the effectiveness of the combination.

DEFINITE PROJECT REPORT FOR BLACKWATER RESERVOIR - MERRIMACK RIVER BASIN FLOOD CONTROL

## APPENDIX B

## HYDRAULICS

1. The hydraulic design of the Blackwater flood control dam presents no unusual features. The dam will be built in two stages, initially for flood control only and subsequently raised to provide for flood control and power storage, hence selection of the outlet capacities required adjustment so that the total reservoir outflow during a flood would not exceed the design discharge in either development. Since it will be practicable to operate the power plant of the final project during floods with a discharge of about 1,000 c.f.s., consideration had to be given this discharge in providing gated capacity that would permit operation to maintain a total reservoir outflow not exceeding the design outflow. Excess gated outlet capacity was desirable in order to expedite emptying the reservoir after the flood. Also, it is desirable to keep the gate sizes the same. After consideration and correlation of the foregoing, a design outflow of 2,000 c.f.s. was selected. This is provided in the initial development with one 3'-6" x 6'-5" ungated outlet and one 3'-6" x 5'-3" gated outlet. Two additional 3'-6" x 5'-3" gated outlets are to be installed initially but not used for flood regulation. These additional outlets are available, however, for use in decreasing the time required to empty the reservoir after floods. In the ultimate development the ungated outlet will be closed off and no longer used. The design discharge of 2,000 c.f.s. will then be attained at El. 584 by using two of the three gated openings. The third gated outlet is then available for additional capacity for emptying purposes. Partial gate opening will not be required either initially or ultimately under this arrangement. Also, in the ultimate plan, the outflow can be held to 2,000 c.f.s. with the power plant in operation by using only one flood control outlet. No stilling basin is required, the outlets discharging into a rock gorge entirely separate from the earth section of the dam. An Ogee free overfall concrete spillway 240 feet long is provided, the capacity being readily computed from the weir formula (see plate 21). The structure is founded on rock with a curved bucket for the high portion of the spillway to deflect the overflow parallel to the stream bed. The gorge below the spillway is narrow, with steep fall, and will pass the spillway design outflow peak without submerging the spillway crest. In order to prevent possible overflow of the narrow discharge channel, some enlargement

by excavation will be provided. The flood reductions at the dam site were obtained by reservoir routing, using inflow hydrographs. The results are shown for the March 1936 flood, the flood based on the September 1932 storm, and the spillway design flood, on plates 24, 29, and 23, respectively.

2. Reductions to the flood at the dam site were carried downstream to the damage centers through Haverhill on the Merrimack River. The March 1936 flood, surpassing all other floods of record in the Merrimack Basin in magnitude and general extent, provides an excellent measure of the probable effectiveness of the reservoir. Downstream reductions were computed for the Blackwater Reservoir alone and in combination with the Franklin Falls Reservoir, and also in combination with the Franklin Falls Reservoir and the Contoocook River Flood Diversion Project. The Blackwater Reservoir, with 6.8 inches of storage, is entirely adequate for all minor floods or spring freshets. Its maximum effectiveness in reduction of large floods may best be gaged by its effect on the 1936 flood. The 1936 flood, as recorded at the U.S. Geological Survey gage near Webster, was routed through the reservoir. Plate 24 shows inflow, outflow, and stage hydrographs for this flood. Pertinent data concerning this flood are as follows:

| | |
|---|---|
| 1936 peak discharge . . . . . . . . . . . | 11,000 c.f.s. |
| Reduced reservoir peak discharge . . . | 1,900 c.f.s. |
| Maximum pool elevation . . . . . . . . . | 560.5 |

3. The maximum capacities of the various outlets and the initial full pool elevation of 566 are as follows:

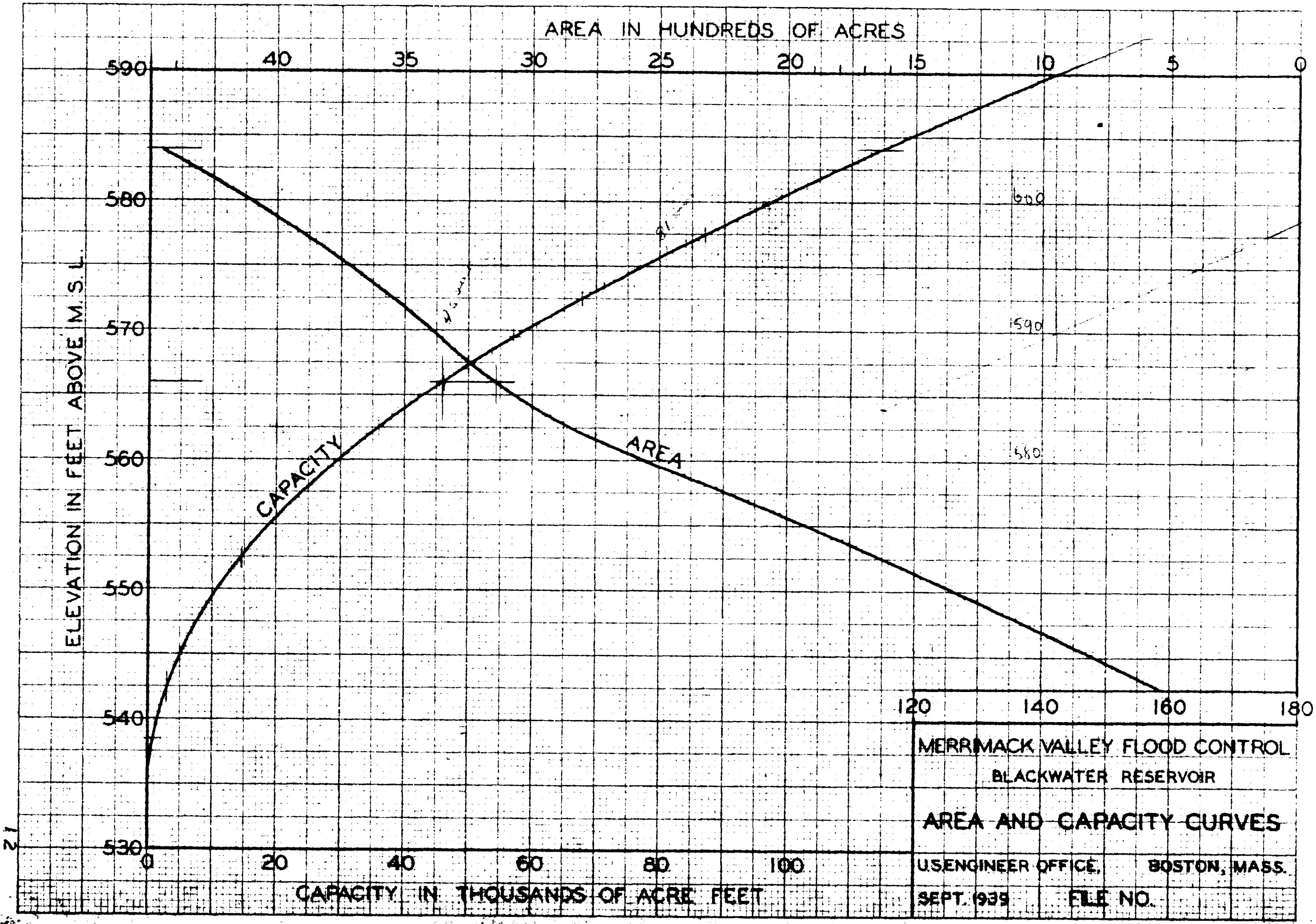
| | |
|---|---|
| One 3'-6" x 6'-5" ungated | - 1,120 c.f.s. |
| Three 3'-6" x 5'-3" gated | - 880 c.f.s. each |

Using these outlets as described in paragraph 1, the 1936 flood was routed through the reservoir. The outflow, reservoir stage and time required for emptying are shown on plate 24. Similar data for the flood based on the 1932 storm are shown on plate 29. The time of emptying from a full pool in the initial plan is 9 days. For the ultimate plan, using three gated outlets with a maximum capacity at El. 584 of 3,000 c.f.s., the time of emptying is about 10 days.

4. The effect of the Blackwater Reservoir in reducing the 1936 flood at two of the downstream damage centers, Manchester, N.H. and Lowell, Mass., on the Merrimack River, is shown by the hydrographs on plates 25 and 26. The same plates also show the discharge reduction credited to the Franklin Falls Reservoir (now under construction) and the combination of the Franklin Falls and Blackwater Reservoirs. It shows at these two points that

Blackwater Reservoir in combination with Franklin Falls adds about 5,000 c.f.s. to the peak reduction for the 1936 flood.

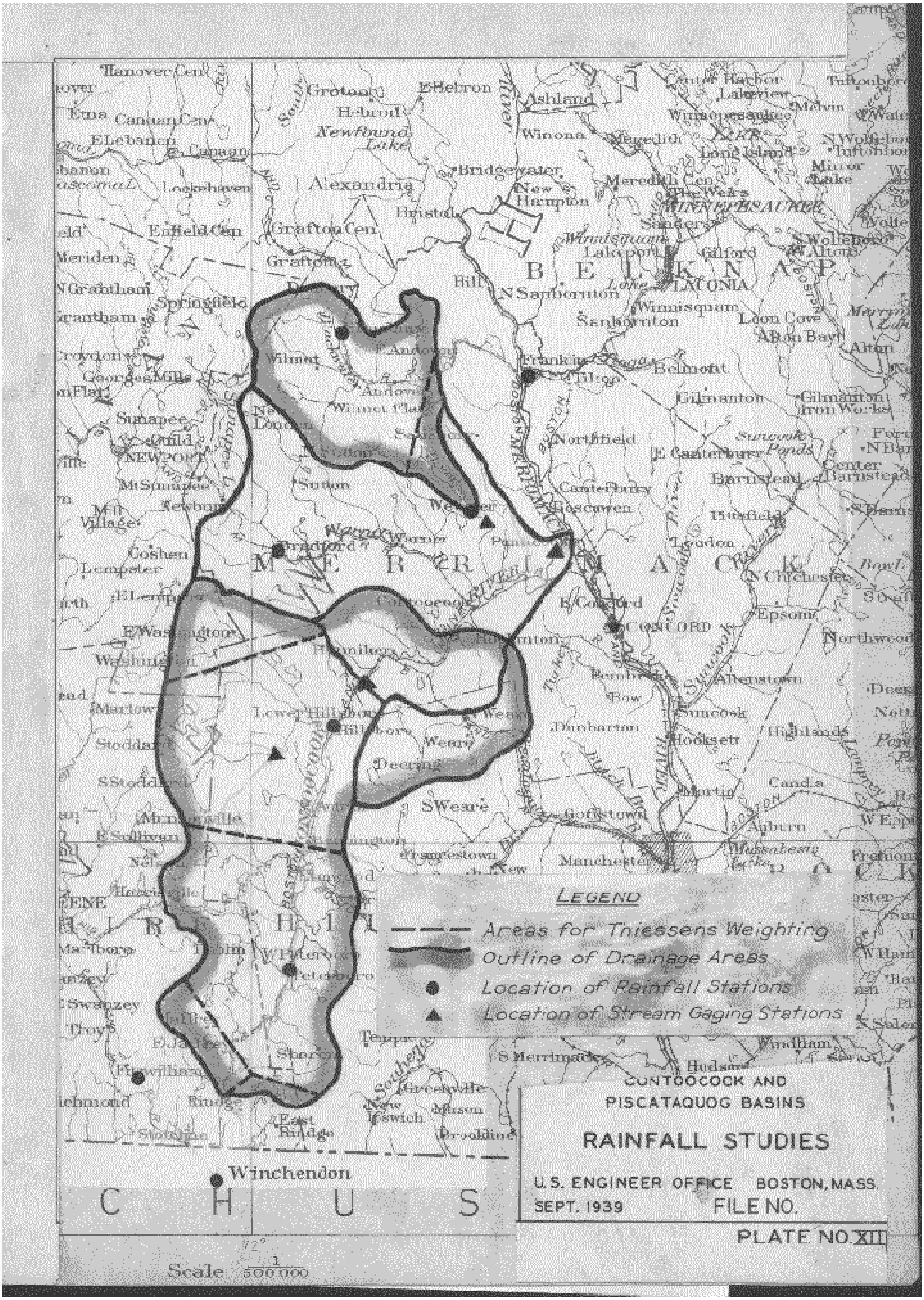
5. Plate 27 shows the effect of distance on the discharge reductions as applied to the flood of March 1936. These curves are based on reach-routing computations and are useful in determining discharge reductions at intermediate damage centers.



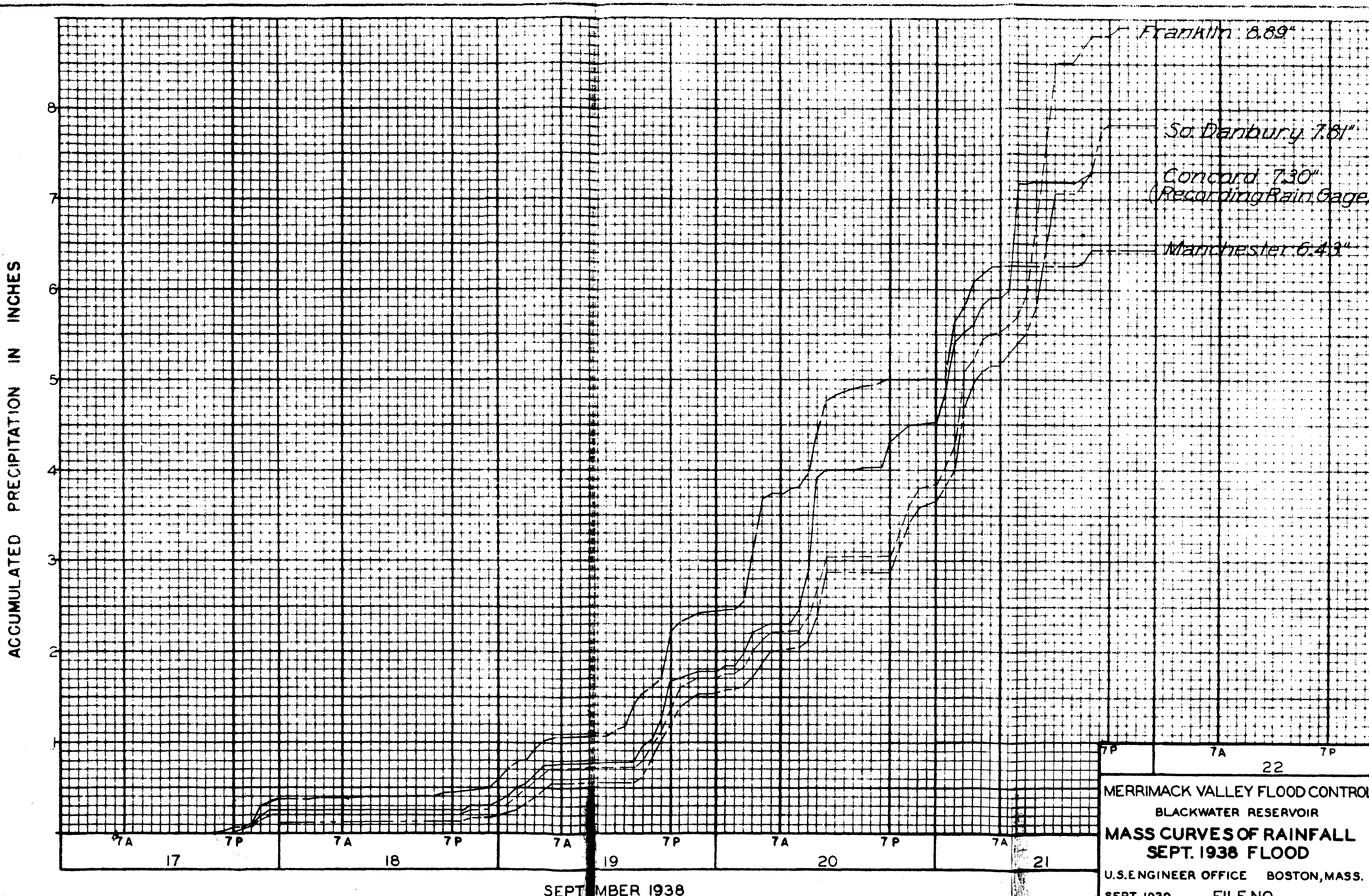

MERRIMACK VALLEY FLOOD CONTROL
BLACKWATER RESERVOIR

AREA AND CAPACITY CURVES

U.S. ENGINEER OFFICE, BOSTON, MASS.
SEPT. 1939 FILE NO.

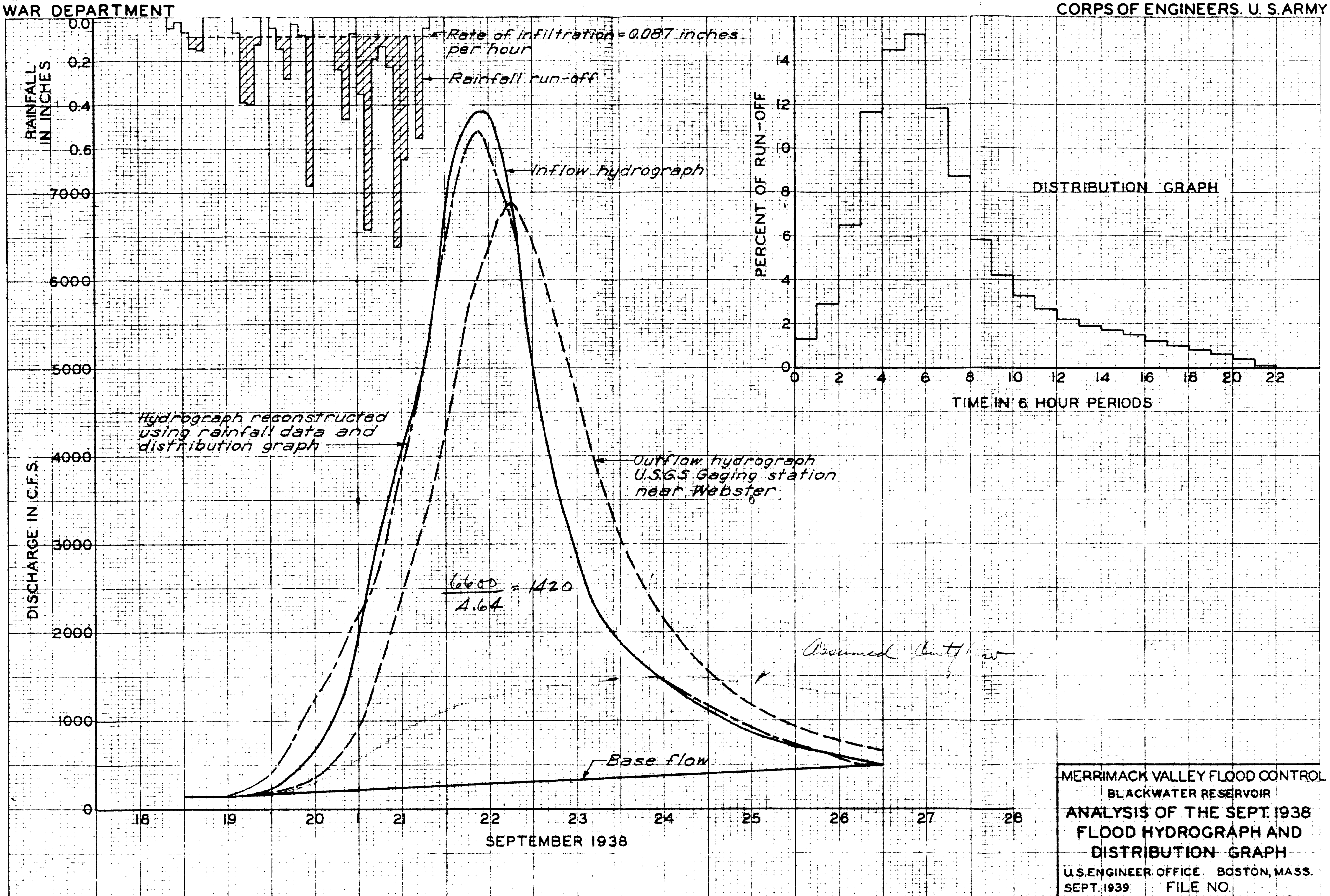


LEGEND
Areas for Thiessens Weighting
Outline of Drainage Areas
Location of Rainfall Stations
Location of Stream Gaging Stations
CONTOOCOOK AND
PISCATAQUOG BASINS
RAINFALL STUDIES
U.S. ENGINEER OFFICE BOSTON, MASS.
SEPT. 1939 FILE NO.
PLATE NO. XII
Scale 1/500000




MERRIMACK VALLEY FLOOD CONTROL
BLACKWATER RESERVOIR
**MASS CURVES OF RAINFALL**
**SEPT. 1938 FLOOD**
U.S. ENGINEER OFFICE BOSTON, MASS.
SEPT. 1939 FILE NO.

14




MERRIMACK VALLEY FLOOD CONTROL
BLACKWATER RESERVOIR

**ANALYSIS OF THE SEPT. 1938 FLOOD HYDROGRAPH AND DISTRIBUTION GRAPH**

U.S. ENGINEER OFFICE BOSTON, MASS.
SEPT. 1939. FILE NO.

WAR DEPARTMENT

U. S. ENGINEER OFFICE, BOSTON, MASS.

Page

Subject: BLACKWATER RESERVOIR

Computation: DISTRIBUTION VALUES BASED ON 1938 FLOOD

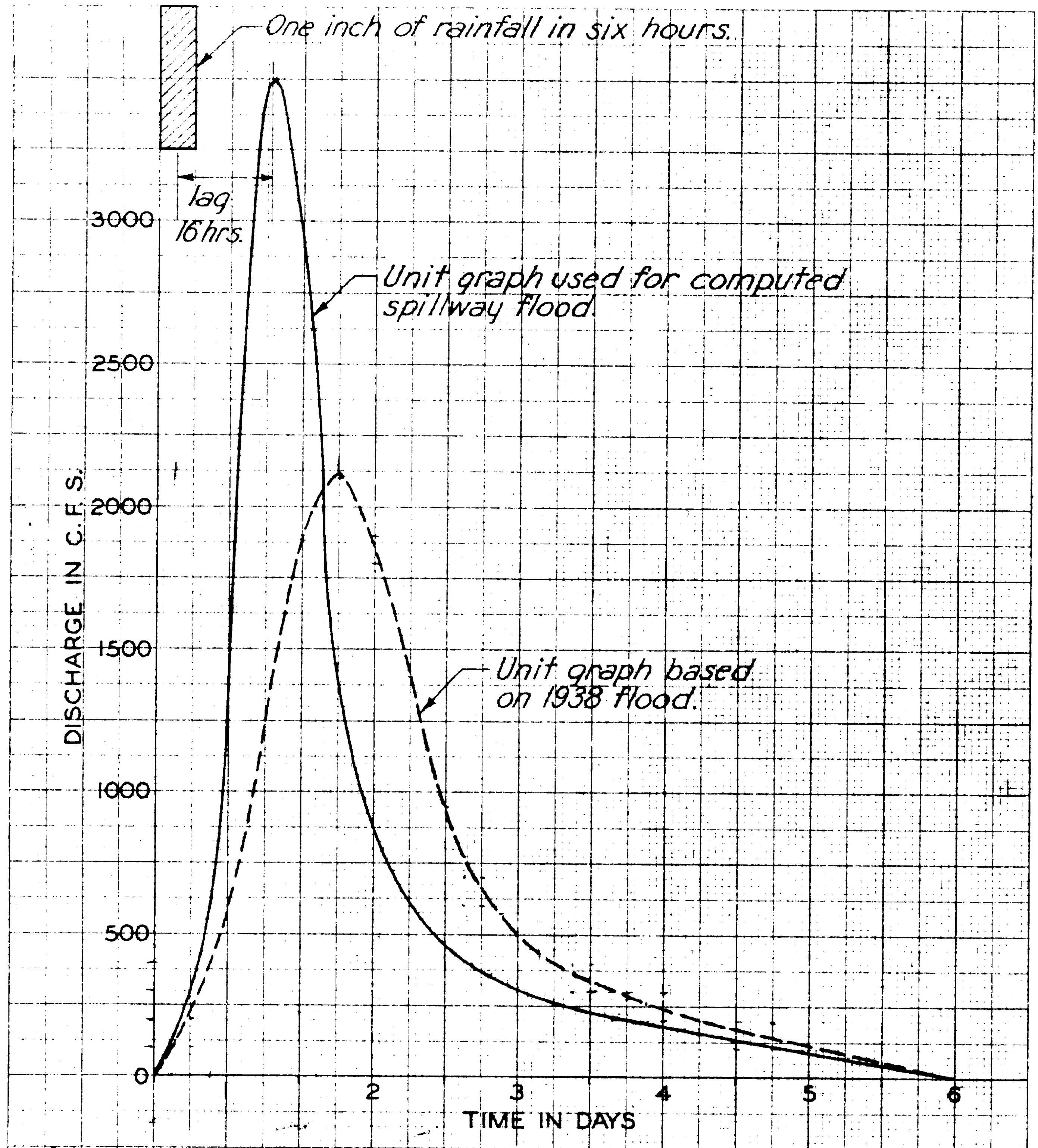
Computed by: E.

Checked by:

Date: 9/2/39

| SEPT. 1938 | RUN-OFF IN. | WAT'D RUN-OFF D.S.F. | TOTAL INFLOW HYDRO. D.S.F. | BASE FLOW D.S.F. | NET INFLOW HYDRO D.S.F. | 0.39/0.39 1.000 | 1.57/0.39 4.02 | 1.04/0.39 2.67 | 0.45/0.39 1.154 | 0.60/0.39 1.540 | 0.14/0.39 0.359 | 0.45/0.39 1.154 | DISTRIBUTION VALUES | PERIODS 6 HOURS |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 19 N | WEIGHTED RAINFALL LESS INFILTRATION | | | | | (ADJUSTED VALUES) | THIS COLUMN USED FOR DERIVATION OF DISTRIBUTION VALUES | | | | | | | |
| 6P | 0.45 | 1537 | 40 | 37 | 3 | | | | | | | 21 | | |
| M | | | 50 | 40 | 10 | | | | | | | 46 | | |
| 6A | 0.14 | 478 | 8[illegible] | 43 | 39 | | | | | | 6 | 104 | | |
| 20 N | 0.60 | 2049 | 137 | 46 | 91 | | | | | 28 | 14 | 186 | | |
| 6P | | | 217 | 49 | 168 | | | | | 62 | 32 | 231 | | |
| M | 0.45 | 1537 | 363 | 52 | 311 | | | | 21 | 139 | 58 | 242 | | |
| 6A | 1.04 | 3551 | 650 | 55 | 595 | | | 48 | 46 | 248 | 72 | 187 | | |
| 21 N | 1.57 | 5361 | 884 | 58 | 830 | | 73 | 107 | 104 | 308 | 75 | 138 | 1.3 | 1 |
| 6P | 0.39 | 1332 | 1140 | 61 | 1079 | (18) 27 | 161 | 240 | 186 | 323 | 58 | 84 | 2.9 | 2 |
| M | | | 1460 | 64 | 1396 | (40) 13 | 362 | 430 | 231 | 250 | 43 | 67 | 6.5 | 3 |
| 6A | | | 1875 | 67 | 1808 | (90) 116 | 650 | 534 | 242 | 185 | 29 | 52 | 11.7 | 4 |
| 22 N | | | 1970 | 70 | 1900 | 161 | 805 | 560 | 187 | 123 | 21 | 43 | 14.5 | 5 |
| 6P | | | 1875 | 73 | 1802 | (200) 247 | 845 | 432 | 138 | 89 | 16 | 35 | 15.2 | 6 |
| M | | | 1500 | 76 | 1424 | (210) 248 | 652 | 320 | 92 | 69 | 13 | 30 | 11.8 | 7 |
| 6A | | | 1090 | 79 | 1019 | 162 | 481 | 214 | 67 | 57 | 11 | 26 | 8.7 | 8 |
| 23 N | | | 835 | 82 | 743 | (120) 136 | 322 | 155 | 52 | 46 | 9 | 23 | 5.8 | 9 |
| 6P | | | 625 | 85 | 540 | (80) 75 | 234 | 120 | 4[illegible] | 40 | 8 | 20 | 4.2 | 10 |
| M | | | 510 | 88 | 422 | (58) 49 | 181 | 99 | 35 | 35 | 7 | 16 | 3.3 | 11 |
| 6A | | | 438 | 91 | 347 | (45) 38 | 149 | 80 | 30 | 31 | 6 | 13 | 2.7 | 12 |
| 24 N | | | 388 | 94 | 294 | 37 | 121 | 70 | 26 | 26 | 5 | 9 | 2.2 | 13 |
| 6P | | | 338 | 97 | 241 | (30) 20 | 105 | 61 | 23 | 22 | 4 | 6 | 1.9 | 14 |
| M | | | 300 | 100 | 200 | (26) 11 | 93 | 53 | 20 | 17 | 3 | 3 | 1.7 | 15 |
| 6A | | | 262 | 103 | 159 | (23) 3 | 81 | 45 | 16 | 12 | 2 | | 1.5 | 16 |
| 25 N | | | 230 | 106 | 124 | (20) -3 | 68 | 37 | 13 | 9 | 1 | | 1.2 | 17 |
| 6P | | | 207 | 109 | 98 | (17) -1 | 56 | 29 | 9 | 5 | | | 1.0 | 18 |
| M | | | 188 | 112 | 76 | (14) 5 | 44 | 21 | 6 | | | | 0.8 | 19 |
| 6A | | | 175 | 115 | 60 | (11) 12 | 32 | 13 | 3 | | | | 0.6 | 20 |
| N | | | 15[illegible] | 118 | 37 | (8) 9 | 20 | 8 | | | | | 0.4 | 21 |
| 6P | | | 142 | 121 | 21 | (5) 9 | 12 | | | | | | 0.1 | 22 |
| M | | | 130 | 124 | 6 | (3) 6 | | | | | | | | |
| | 4.64 | 15845 | | | 16,843 | | 5378 | | | | | | 100.0 | |

Plate [illegible] 1[illegible]




MERRIMACK VALLEY FLOOD CONTROL

BLACKWATER RESERVOIR

**INFLOW UNIT HYDROGRAPH**

U.S. ENGINEER OFFICE, BOSTON, MASS.

SEPT, 1939. FILE NO.

PLATE NO. XVII

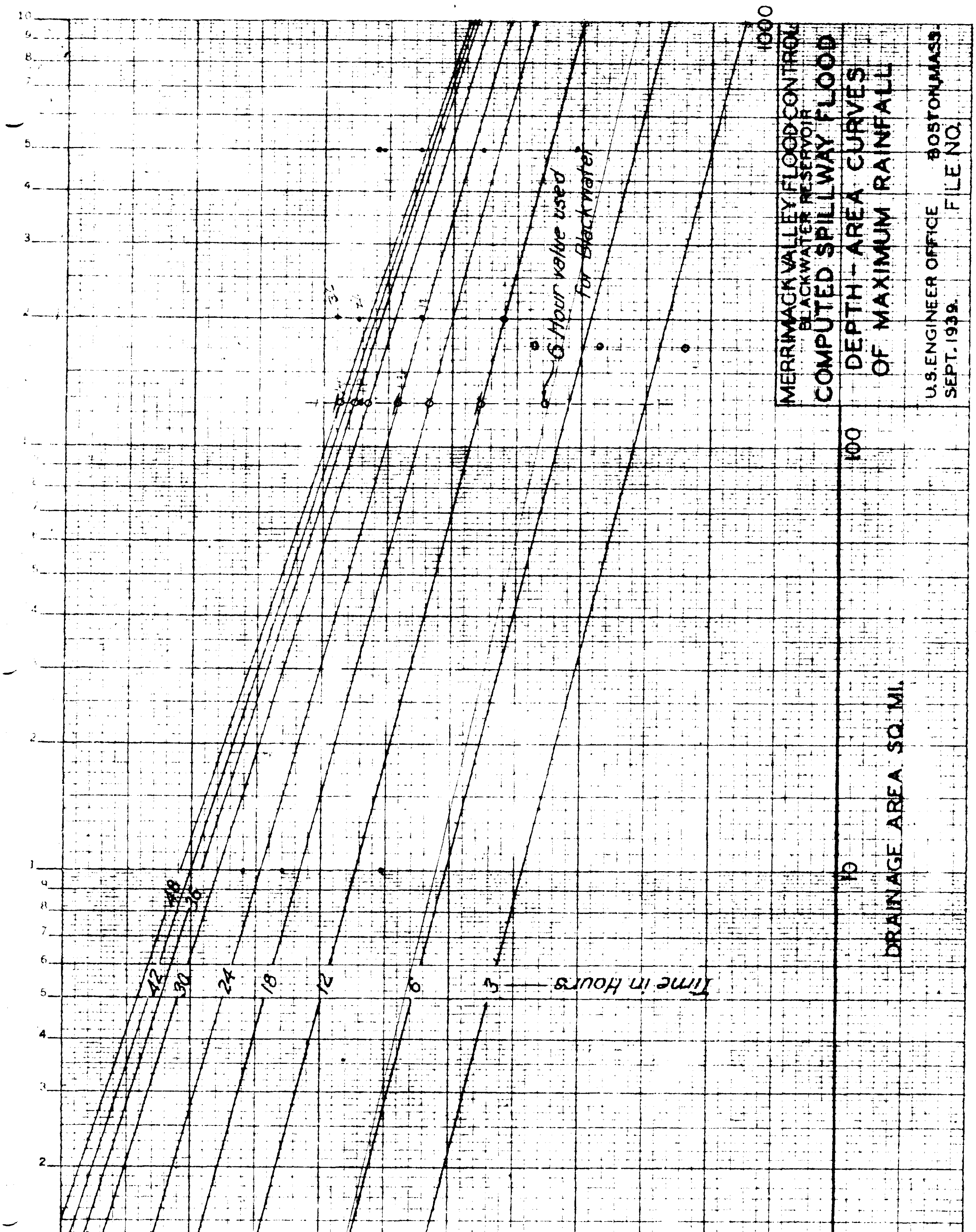



PLATE NO. XVIII

nd officer/board, and make recommendations to

recommendations of the surveyin

# WAR DEPARTMENT

U. S. ENGINEER OFFICE, BOSTON, MASS.

Page

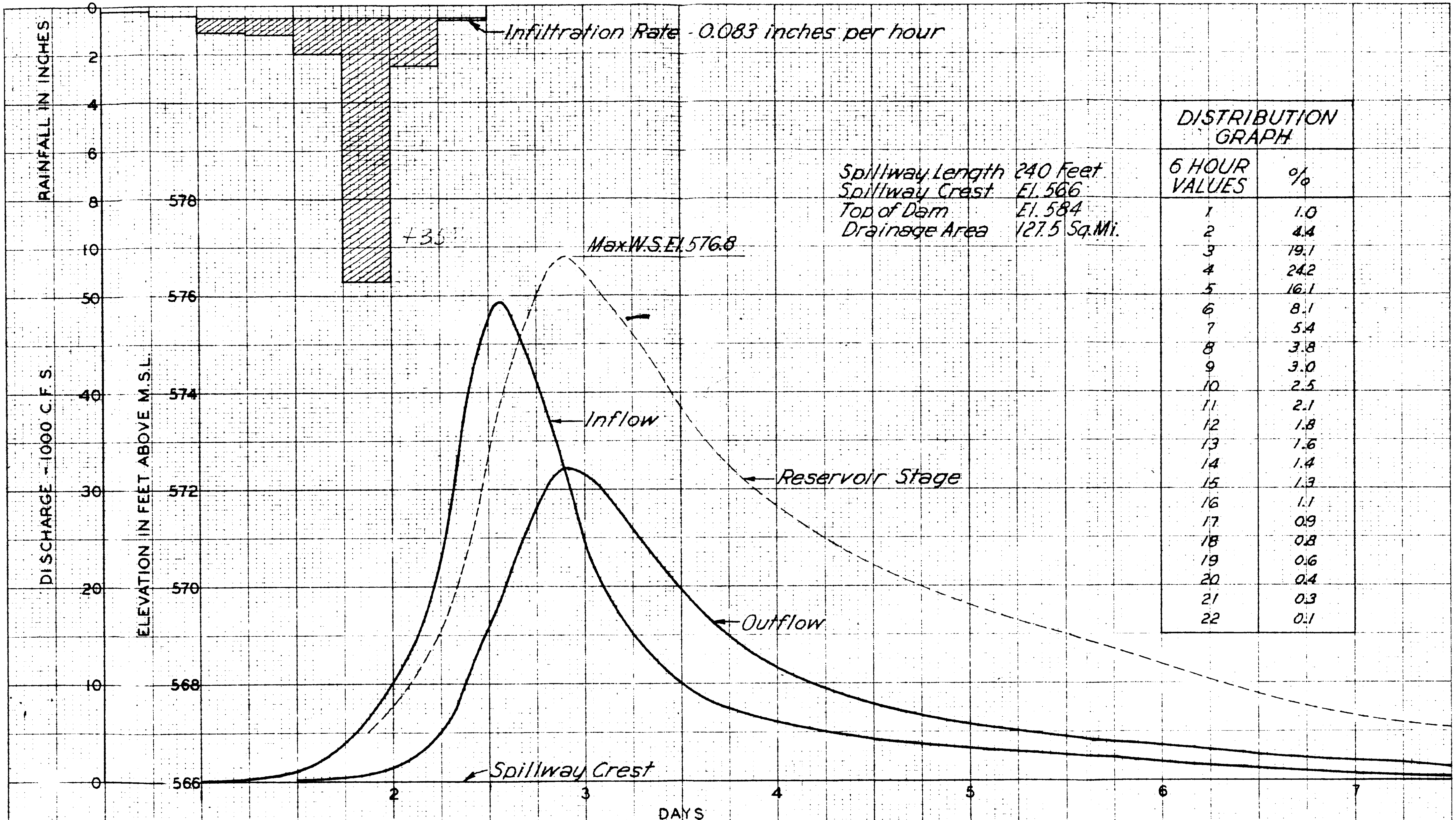
Subject: BLACKWATER RESERVOIR

Computation: COMPUTED SPILLWAY FLOOD

Computed by: FCM    Checked by: W.M.W.    Date: 11/13/39

| Period | Rainfall inches | Runoff (Infiltrate .083"/hr) inches | Run-off d.s.f | Distribution Values % | Distributed Run-off – d.s.f. 1 | 2 | 3 | 4 | 5 | 6 | Total Run-off d.s.f | Total Run-off d.s.f/¼ day | Total Runoff with Base Flow – 2 cfs per sq mi d.s.f/¼ day |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| . M | 0.0 | 0.0 | | | | | | | | | | | |
| 6A | 0.2 | 0.0 | | | | | | | | | | | |
| 1 N | 0.4 | 0.0 | | | | | | | | | | | |
| 6P | 1.1 | 0.6 | 2057 | 1.0 | 21 | | | | | | 21 | 84 | 339 |
| M | 1.2 | 0.7 | 2400 | 4.4 | 91 | 24 | | | | | 105 | 460 | 715 |
| 6A | 2.0 | 1.5 | 5143 | 19.1 | 393 | 106 | 51 | | | | 550 | 2200 | 2455 |
| 2 N | 11.4 | 10.9 | 37400 | 24.2 | 498 | 458 | 226 | 374 | | | 1556 | 6224 | 6479 |
| 6P | 2.5 | 2.0 | 6860 | 16.1 | 331 | 581 | 982 | 1645 | 68 | | 3607 | 14428 | 14683 |
| M | 0.6 | 0.1 | 343 | 8.1 | 167 | 386 | 1245 | 7140 | 302 | 3 | 9243 | 36972 | 37227 |
| 6A | | | | 5.4 | 111 | 194 | 828 | 9050 | 1310 | 15 | 11508 | 46032 | 46287 |
| 3 N | | | | 3.8 | 78 | 130 | 417 | 6020 | 1665 | 66 | 8376 | 33504 | 33759 |
| 6P | | | | 3.0 | 62 | 91 | 278 | 3025 | 1110 | 83 | 4649 | 18596 | 18851 |
| M | | | | 2.5 | 51 | 72 | 195 | 2020 | 556 | 55 | 2949 | 11796 | 12051 |
| 6A | | | | 2.1 | 43 | 60 | 154 | 1422 | 371 | 28 | 2078 | 8312 | 8567 |
| 4 N | | | | 1.8 | 37 | 50 | 129 | 1120 | 262 | 19 | 1617 | 6468 | 6723 |
| 6P | | | | 1.6 | 33 | 43 | 108 | 935 | 206 | 13 | 1338 | 5352 | 5607 |
| M | | | | 1.4 | 29 | 38 | 93 | 785 | 172 | 10 | 1127 | 4508 | 4763 |
| 6A | | | | 1.3 | 27 | 34 | 82 | 673 | 144 | 9 | 969 | 3876 | 4131 |
| 5 N | | | | 1.1 | 23 | 31 | 72 | 598 | 124 | 7 | 855 | 3420 | 3675 |
| 6P | | | | 0.9 | 19 | 26 | 67 | 524 | 110 | 6 | 752 | 3008 | 3263 |
| M | | | | 0.8 | 16 | 22 | 57 | 486 | 96 | 5 | 682 | 2728 | 2983 |
| 6A | | | | 0.6 | 12 | 19 | 46 | 411 | 89 | 5 | 582 | 2328 | 2583 |
| 6 N | | | | 0.4 | 8 | 14 | 41 | 337 | 73 | 4 | 479 | 1916 | 2171 |
| 6P | | | | 0.3 | 6 | 10 | 31 | 299 | 62 | 4 | 412 | 1648 | 1903 |
| M | | | | 0.1 | 2 | 7 | 21 | 224 | 55 | 3 | 312 | 1248 | 1503 |
| | | | | | | 2 | 15 | 149 | 41 | 3 | 210 | 840 | 1095 |
| | | | | | | | 5 | 112 | 28 | 2 | 147 | 588 | 843 |
| | | | | | | | | 37 | 21 | 1 | 59 | 236 | 491 |
| | | | | | | | | | 7 | 1 | 8 | 32 | 287 |
| | 19.4 | 15.8 | 54203 | 100.0 | 2058 | 2398 | 5143 | 37386 | 6874 | 342 | 54191 | 216804 | 223434 |

PLATE NO. XIX




| DISTRIBUTION GRAPH | |
|---|---|
| 6 HOUR VALUES | % |
| 1 | 1.0 |
| 2 | 4.4 |
| 3 | 19.1 |
| 4 | 24.2 |
| 5 | 16.1 |
| 6 | 8.1 |
| 7 | 5.4 |
| 8 | 3.8 |
| 9 | 3.0 |
| 10 | 2.5 |
| 11 | 2.1 |
| 12 | 1.8 |
| 13 | 1.6 |
| 14 | 1.4 |
| 15 | 1.3 |
| 16 | 1.1 |
| 17 | 0.9 |
| 18 | 0.8 |
| 19 | 0.6 |
| 20 | 0.4 |
| 21 | 0.3 |
| 22 | 0.1 |

MERRIMACK VALLEY FLOOD CONTROL
BLACKWATER RESERVOIR

**COMPUTED SPILLWAY FLOOD INFLOW - OUTFLOW STAGE HYDROGRAPH**

U.S. ENGINEER OFFICE BOSTON MASS.
SEPT. 1939 FILE NO.

PLATE NO. XX

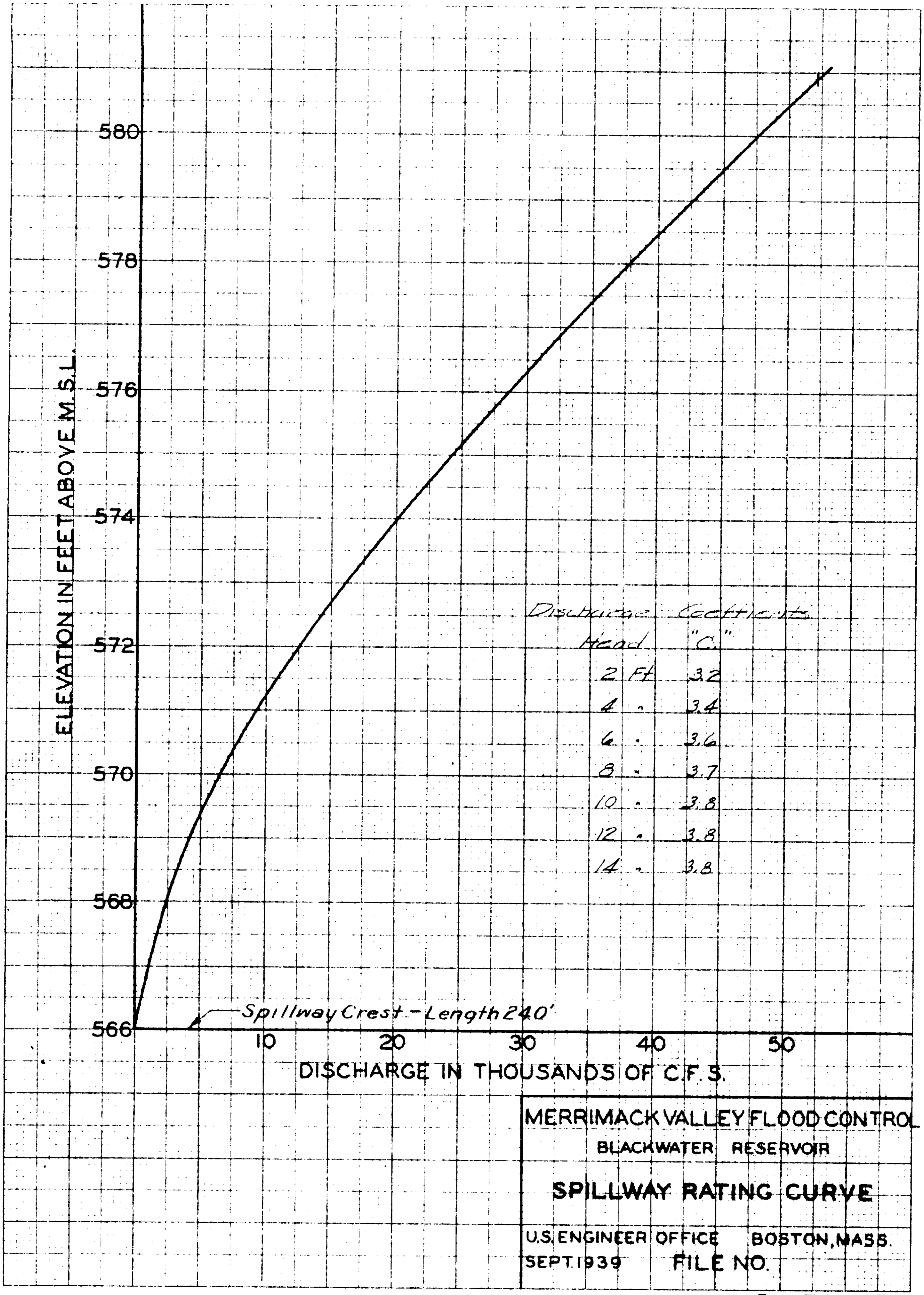



PLATE NO. XXI

minimum 77 percent of base in bearing; and the maximum

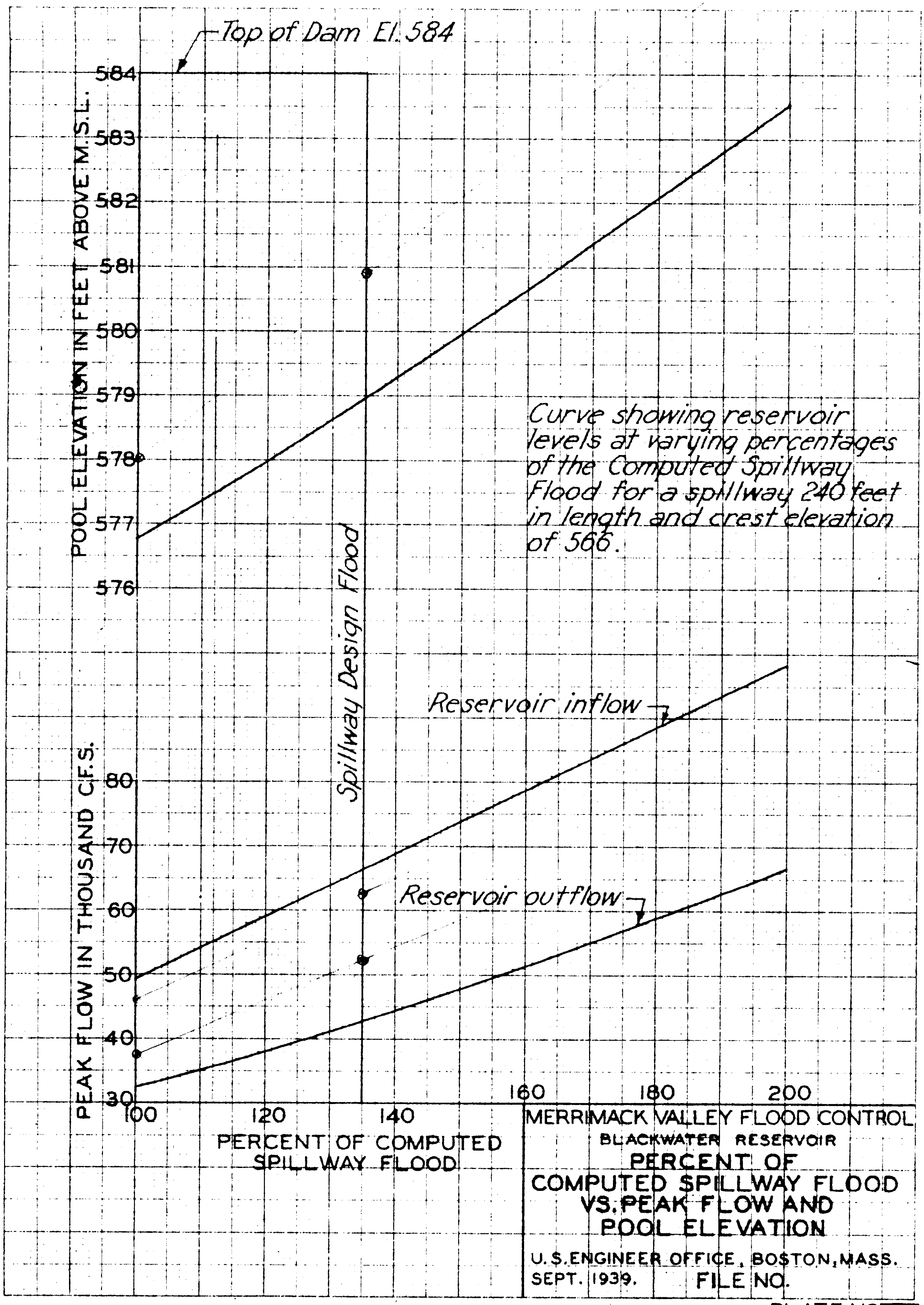
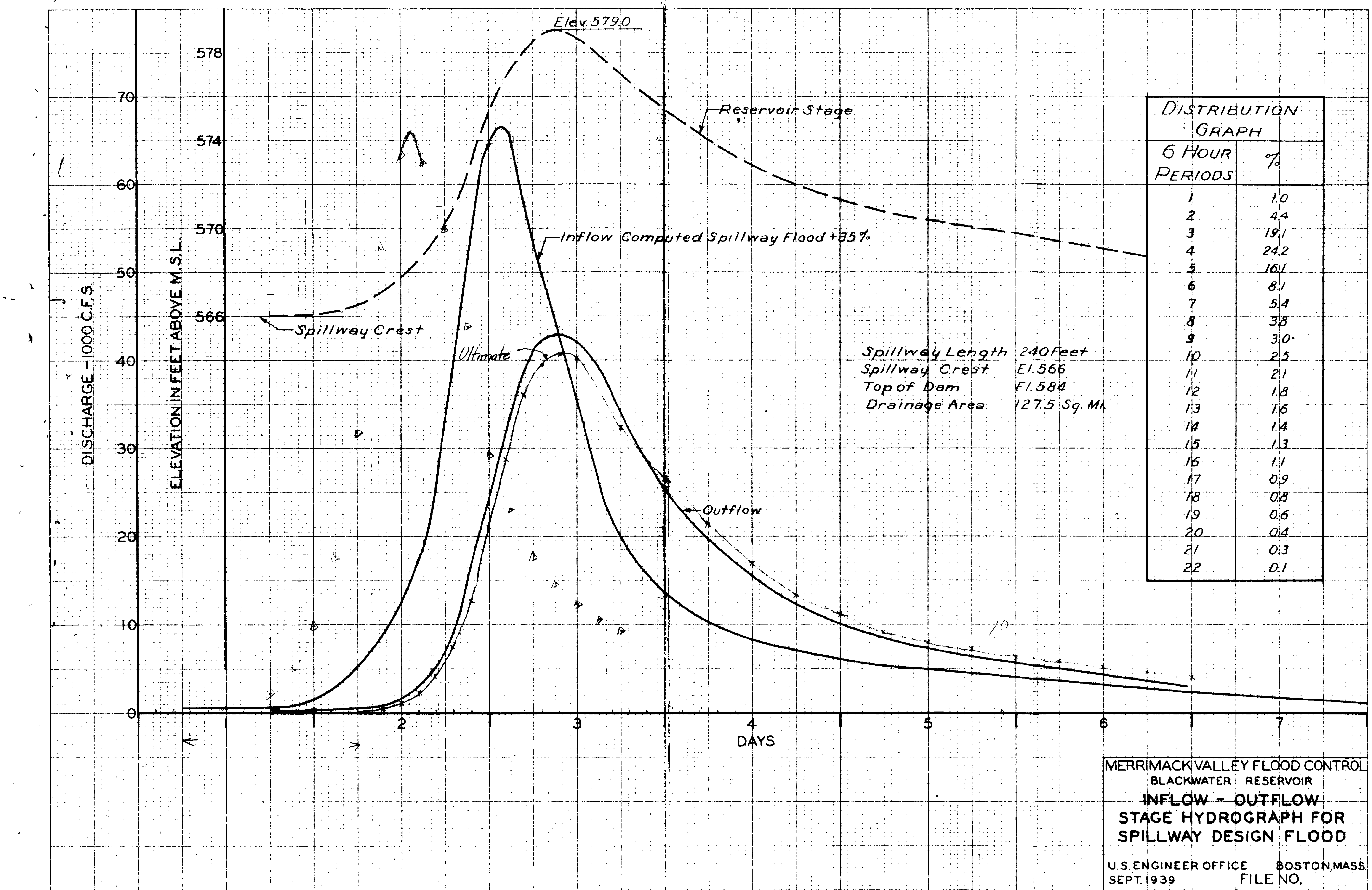



PLATE NO XXII



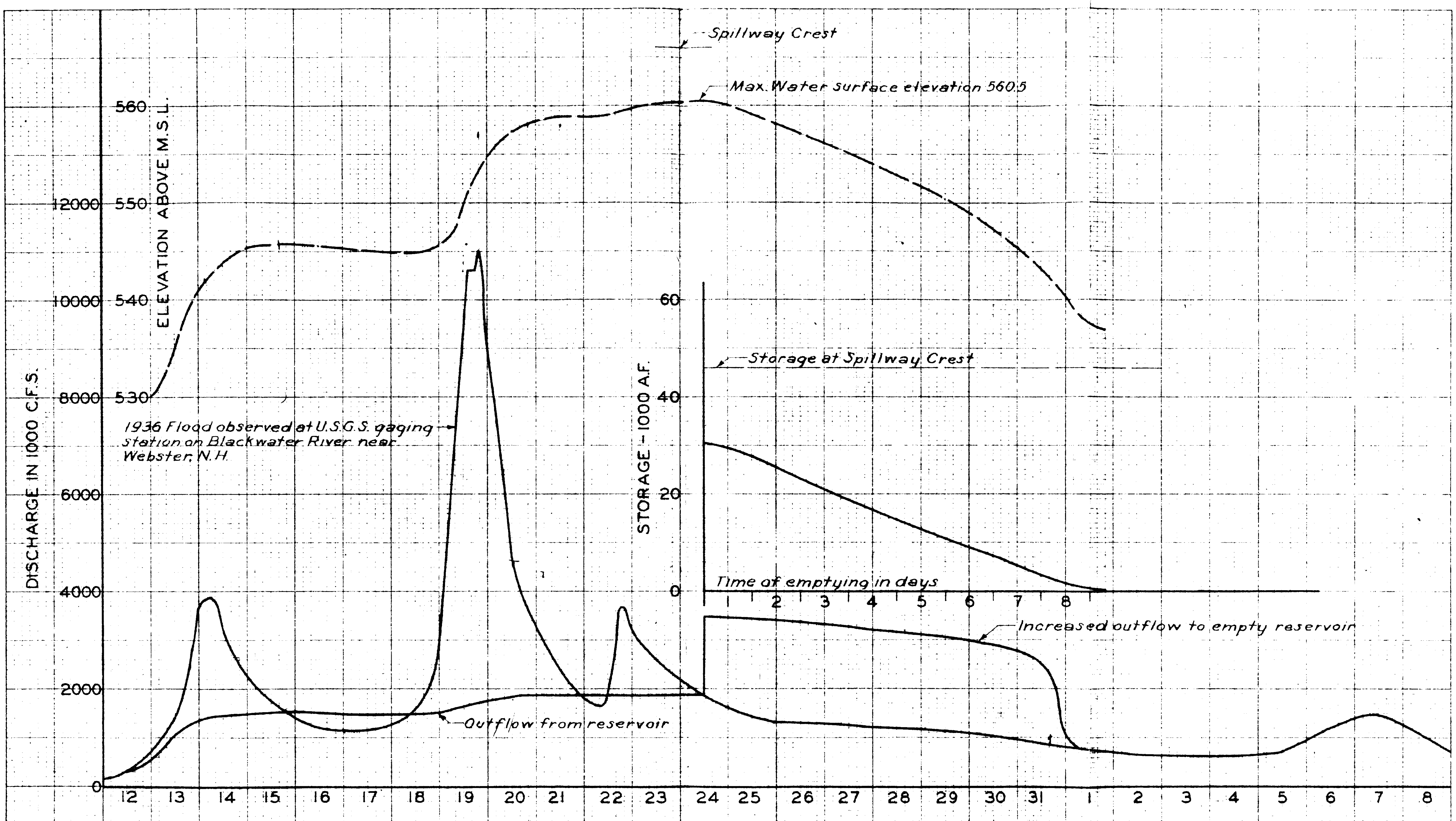

| DISTRIBUTION GRAPH | |
|---|---|
| 6 HOUR PERIODS | % |
| 1 | 1.0 |
| 2 | 4.4 |
| 3 | 19.1 |
| 4 | 24.2 |
| 5 | 16.1 |
| 6 | 8.1 |
| 7 | 5.4 |
| 8 | 3.8 |
| 9 | 3.0 |
| 10 | 2.5 |
| 11 | 2.1 |
| 12 | 1.8 |
| 13 | 1.6 |
| 14 | 1.4 |
| 15 | 1.3 |
| 16 | 1.1 |
| 17 | 0.9 |
| 18 | 0.8 |
| 19 | 0.6 |
| 20 | 0.4 |
| 21 | 0.3 |
| 22 | 0.1 |

MERRIMACK VALLEY FLOOD CONTROL
BLACKWATER RESERVOIR

**INFLOW - OUTFLOW**
**STAGE HYDROGRAPH FOR**
**SPILLWAY DESIGN FLOOD**

U.S. ENGINEER OFFICE BOSTON, MASS.
SEPT. 1939 FILE NO.

PLATE NO. XXIII




MERRIMACK VALLEY FLOOD CONTROL

**EFFECT OF BLACKWATER RESERVOIR ON THE 1936 FLOOD**

U S ENGINEER OFFICE BOSTON, MASS.

SEPT. 1939 FILE NO.

PLATE NO. XXIV

Fig 2

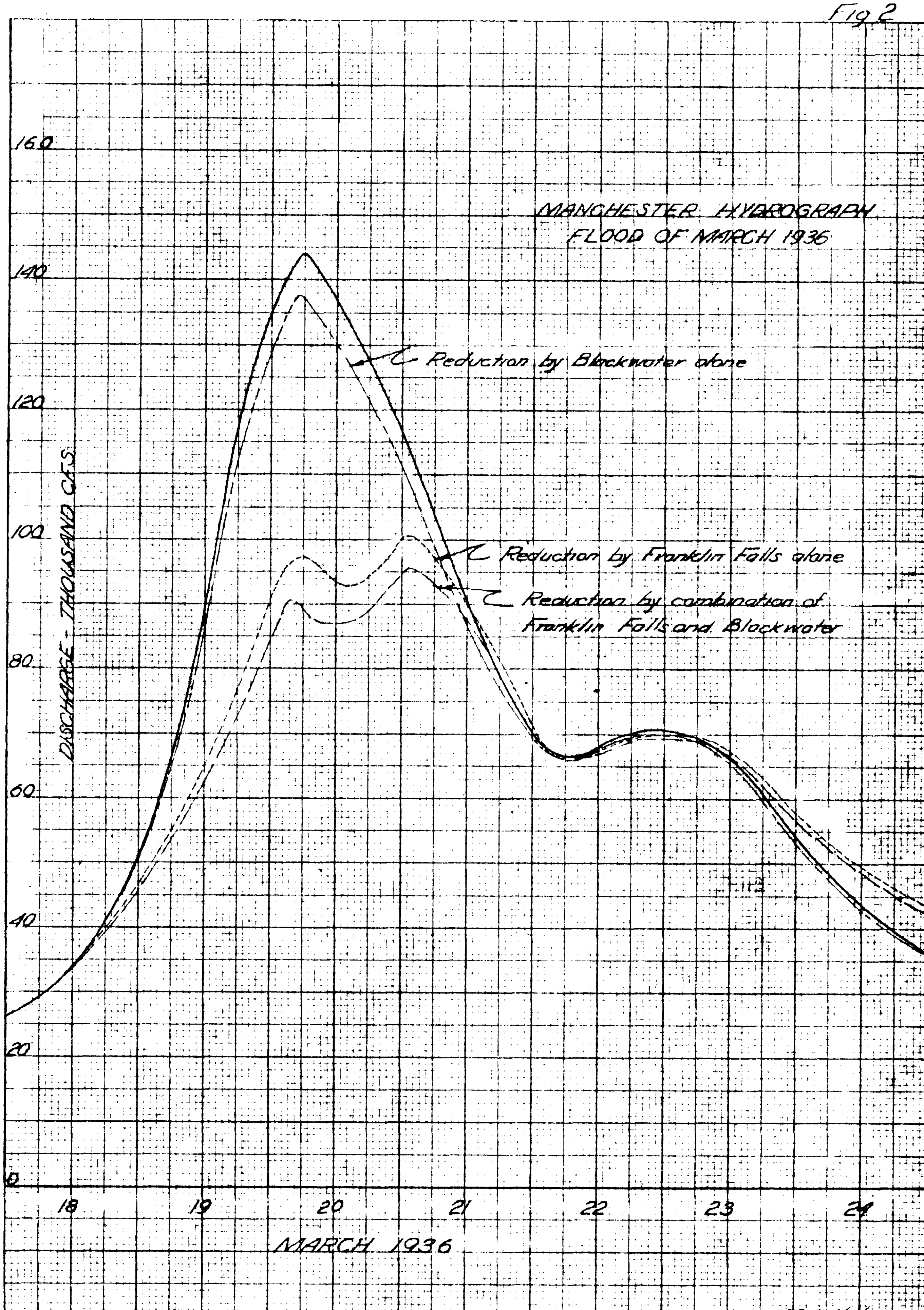



Plate 25

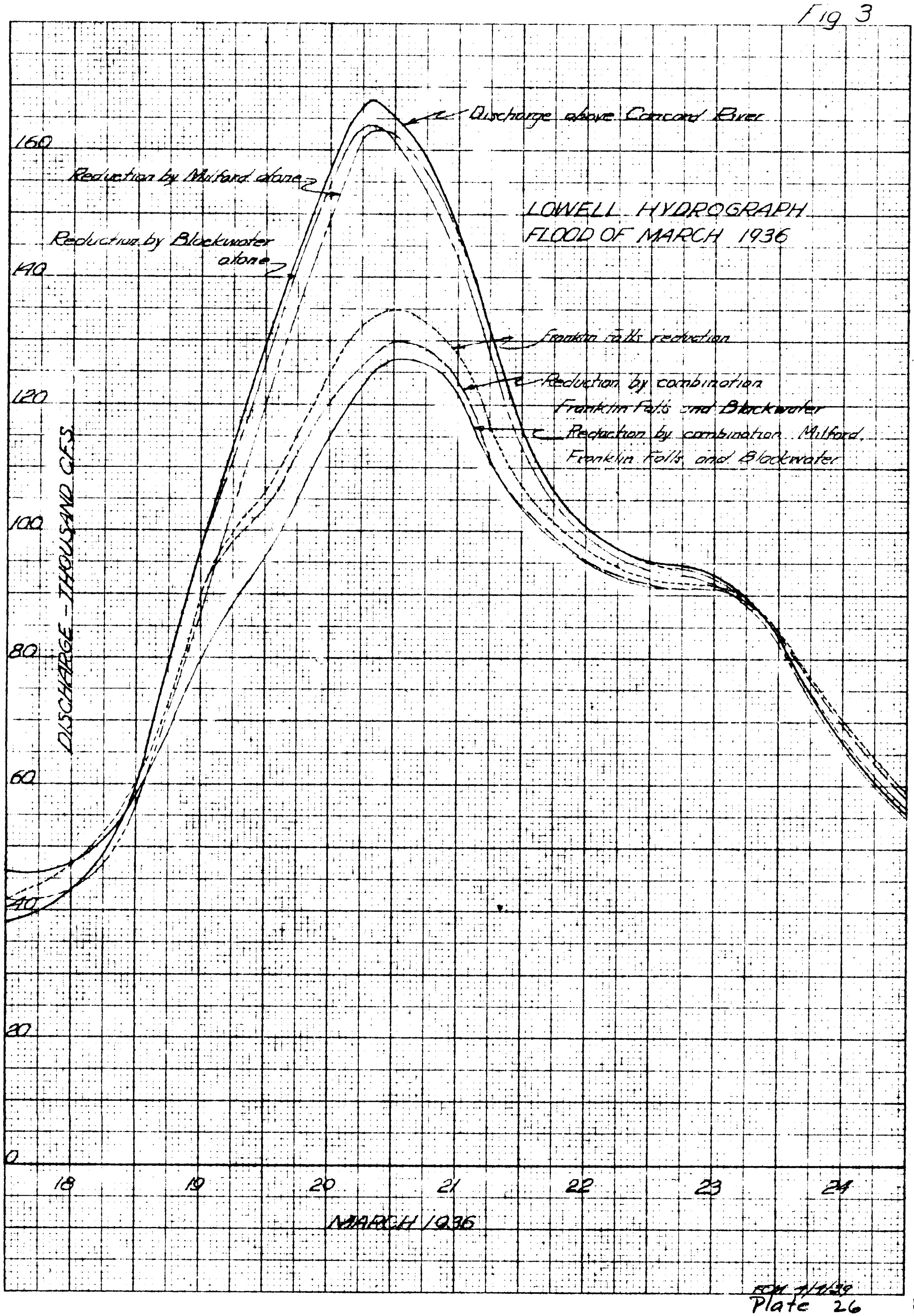


Fig 3
LOWELL HYDROGRAPH
FLOOD OF MARCH 1936
Discharge above Concord River
Reduction by Milford alone
Reduction by Blackwater alone
Franklin Falls reduction
Reduction by combination Franklin Falls and Blackwater
Reduction by combination Milford, Franklin Falls and Blackwater
DISCHARGE - THOUSAND C.F.S.
160
140
120
100
80
60
40
20
0
18
19
20
21
22
23
24
MARCH 1936
Plate 26

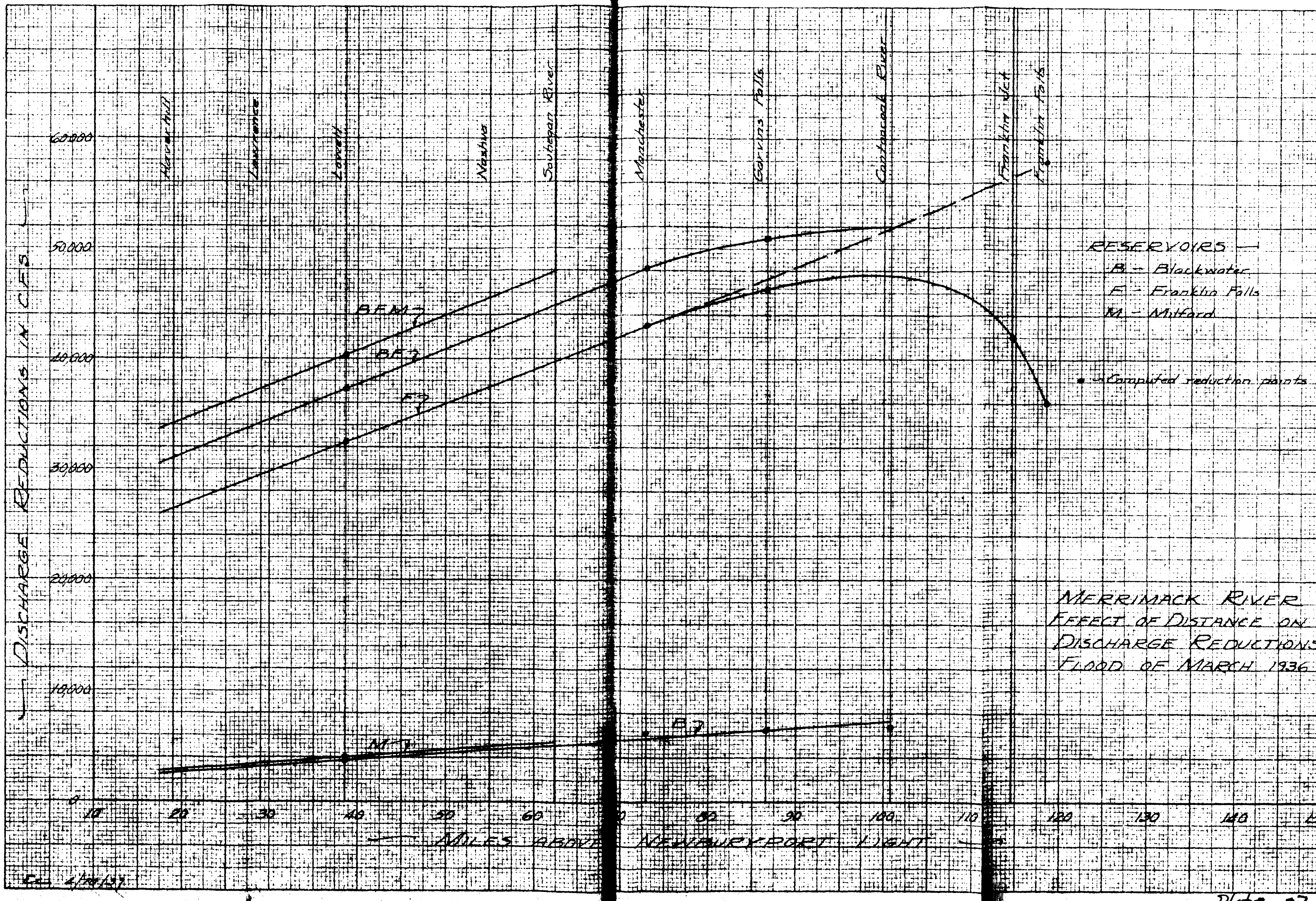
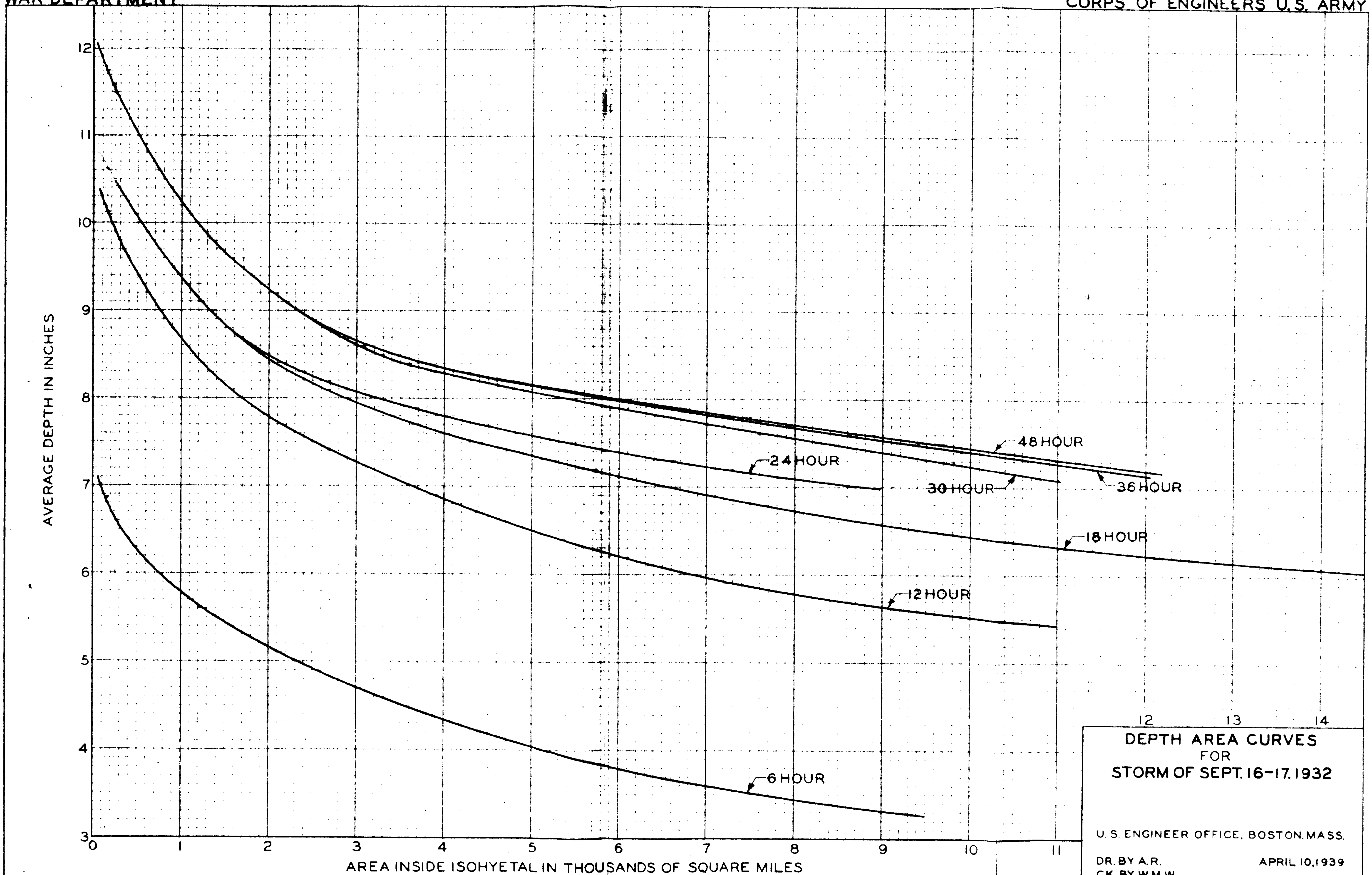

Haverhill
Lawrence
Lowell
Nashua
Souhegan River
Manchester
Garvins Falls
Contoocook River
Franklin Jct.
Franklin Falls
60,000
50,000
40,000
30,000
20,000
10,000
0
10
20
30
40
50
60
70
80
90
100
110
120
130
140
150
BFM
BF
F
B
M
DISCHARGE REDUCTIONS IN C.F.S.
MILES ABOVE NEWBURYPORT LIGHT
RESERVOIRS —
B - Blackwater
F - Franklin Falls
M - Milford
• - Computed reduction points
MERRIMACK RIVER
EFFECT OF DISTANCE ON
DISCHARGE REDUCTIONS
FLOOD OF MARCH 1936

Plate 27

WAR DEPARTMENT

CORPS OF ENGINEERS U.S. ARMY



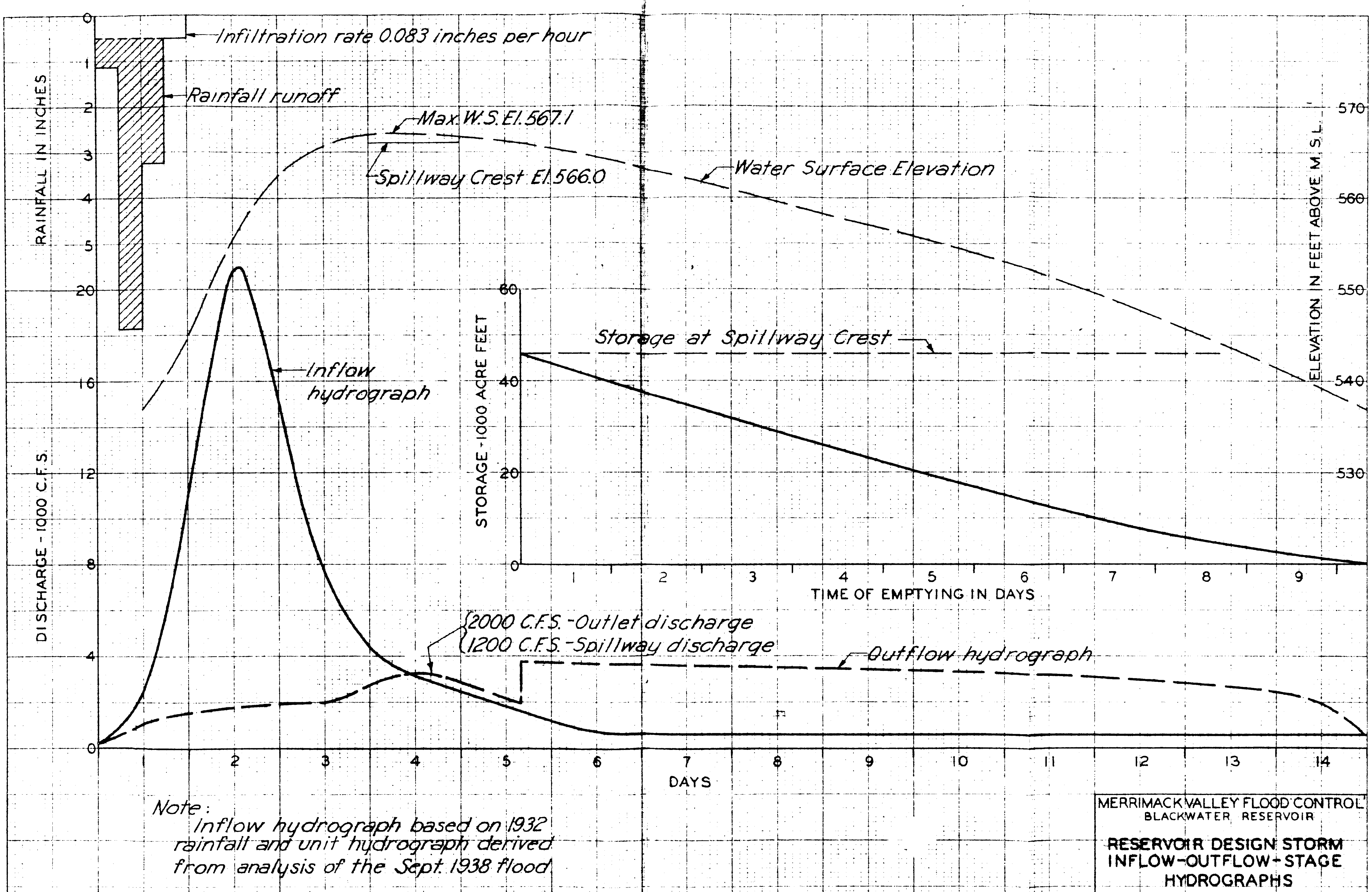

DEPTH AREA CURVES
FOR
STORM OF SEPT. 16-17, 1932

U.S. ENGINEER OFFICE, BOSTON, MASS.

DR. BY A.R.
CK. BY W.M.W.

APRIL 10, 1939

PLATE NO. XXVIII.




MERRIMACK VALLEY FLOOD CONTROL
BLACKWATER RESERVOIR

**RESERVOIR DESIGN STORM INFLOW-OUTFLOW-STAGE HYDROGRAPHS**

U.S. ENGINEER OFFICE BOSTON MASS.
SEPT. 1939 FILE NO.

PLATE NO. XXIX

A N A L Y S I S   O F   D E S I G N

B L A C K W A T E R   D A M

## Table of Contents

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Table of Contents (Cont'd.)



Figures

## Table of Contents (Cont'd.)

Table of Contents (Cont'd.)



(To be made part of the Analysis of Design after construction of the project is complete)

APPENDICES (in separate volumes)

| Letter | | Title |
|---|---|---|
| A | - | Structural Design Computations |
| B | - | Compaction Tests and Critical Density Investigation of Earth Embankment Materials |
| C | - | Design Conference, November 10, 1939 - Synopsis and Folio |

3-01. Hydrology Report. As required by paragraph 19 of Engineer Bulletin, R. & H. No. 9, 1938, a hydrology report was submitted for approval as part of the Definite Project Report for Blackwater Reservoir, dated September 15, 1939 (E.D. File 7402(Merrimack R.-Blackwater Res.) -8). The Definite Project Report was approved by the Chief of Engineers subject to further hydrological studies. The results of these additional studies were submitted in the form of revised sections of the Definite Project Report on December 16, 1939. The following paragraphs on the hydrology and hydraulic factors affecting the design of the Blackwater Project constitute, therefore, a summary of the hydrology studies referred to above.

3-02. General Hydrology. The mean annual precipitation is about 39 inches, fairly evenly distributed throughout the year. Precipitation during the months of December, January, February and March is principally in the form of snow which may accumulate for most of the winter since temperatures generally remain below freezing during this period. Hence, this period has low run-off and is free from flood threats. Because of snow accumulation, however, Blackwater River is subject to heavy run-off during the spring thaw. Records show that the "spring flood" generally occurs during March or April, although one moderate freshet affected by snow run-off is recorded in the middle of January (January 1935). The high temperature causing snow melt is usually accompanied by rain and is the most frequent cause of floods. The maximum flood of record, that of March 1936, was caused primarily by snow run-off. For the remainder of the year the basin is subject to occasional heavy rainfall from general summer and fall storms. The drainage area is small enough to be seriously affected by intense local storms. The flood produced by the summer storm of September 1938 was the second largest of record.

3-03. Precipitation. (a) Records. - The precipitation observation stations in and adjacent to the Blackwater River are shown on Figure III-1. General data on these stations are listed in the following table:

TABLE III-1 - PRECIPITATION RECORDS

| Name & Location of Station | Elevation in feet Above MSL | Length of Record (Years) | Annual Precipitation in Inches: Mean | Maximum: Amount | Maximum: Year | Minimum: Amount | Minimum: Year |
|---|---|---|---|---|---|---|---|
| So. Danbury (in basin) | 724 | 2 | 49.4 | 51.5 | 1937 | 47.3 | 1938 |
| Webster (in basin) | 550 | 2 | 51.6 | 53.0 | 1938 | 50.3 | 1937 |
| Grafton (10 miles N.)* | 863 | 23 | 38.6 | 50.7 | 1902 | 30.0 | 1908 |
| Franklin (10 miles E.) | 390 | 37 | 40.2 | 49.7 | 1920 | 30.9 | 1930 |
| Concord (20 miles SE.)** | 350 | 86 | 36.4 | 54.3 | 1863 | 26.1 | 1930 |

* U.S. Weather Bureau Station with fragmentary records from 1879 to 1917.
** Continuous recording gage.

(b) Rainfall Intensity. - The maximum rainfall recorded over the entire Blackwater watershed for a given storm was 7.8 inches for the period September 18 to 21, 1938. The maximum rainfall at a single station was 8.8 inches during the same period with a maximum intensity of 4.4 inches in 24 hours on September 21. Depth-duration curves for rainfall at stations in and near the Blackwater area were constructed in conformity with the continuously recorded data available for the Concord, N. H. Station.

(c) Selection of Maximum Precipitation Values. - The U. S. Weather Bureau, cooperating with the Department, has prepared depth-area curves of maximum rainfall considered applicable to the New England area. Values from these curves have been used for the Blackwater Reservoir except that a higher value has been used for a 6-hour period upon recommendation of the Hydrometeorological Section of the Office, Chief of Engineers. The selected values, which are intended for use only as maximum conditions for spillway design purposes, are as follows:

TABLE III-2 - MAXIMUM PRECIPITATION VALUES

| | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| Period in Days | 1/4 | 1/2 | 3/4 | 1 | 1-1/4 | 1-1/2 | 1-3/4 | 2 |
| Period in Hours | 6 | 12 | 18 | 24 | 30 | 36 | 42 | 48 |
| Precipitation, Inches | 11.4 | 13.9 | 15.9 | 17.1 | 18.2 | 18.8 | 19.2 | 19.4 |

3-04. Run-off. (a) Records. - A record of stream flow is available at the U. S. Geological Survey Station, located about two miles downstream from the damsite. Two major floods have occurred during this period of record: In March 1936, peak flow 11,000 c.f.s.; in September 1938, peak flow 6880 c.f.s. The November 1927 flood peak flow was 2620 c.f.s. General data on this record is shown in the following table.

TABLE III-3 - STREAM FLOW RECORDS, BLACKWATER RIVER

| Station | Drainage Area: Sq. Mi. | Period of: Record | Discharge in Cu. Ft. per Sec.: Mean | Maximum | Minimum |
|---|---|---|---|---|---|
| Original location near Contoocook | 134 | 1918-1920<br>1927-1935 | | | |
| Present location near Webster Recording gage | 129 | 1935-1938 | 255 | 11,000 | 20 |

(b) Frequency of Floods. - The stream flow records on Blackwater River are too short for use in computing a satisfactory frequency relation. By comparison with computed discharge frequencies for watersheds of similar size, it appears that the peak flows of 11,000 c.f.s. in March 1936 and 6880 c.f.s. in September 1938 have a frequency of about 106 and 19 years, respectively. Similarly, it appears that flows of 70 and 85 c.f.s. per square mile have a frequency of about once in 50 and 100 years, respectively.

(c) Unit Hydrographs. - The maximum flow of record uninfluenced by snow run-off was 6880 c.f.s. in September 1938. This flood was used for derivation of run-off distribution values. The resulting unit graph, shown as a dotted line on Figure III-2, was used in computations of reservoir design floods. However, for spillway design purposes where peak values are important, it was considered desirable to use a higher peaked unit graph, as shown in solid line on Figure III-2.

3-05. Reservoir Capacity. (a) Flood Control. - Area and capacity curves are shown on Figure III-3. Based on considerations discussed in following paragraphs, a capacity of 46,000 acre-feet, equivalent to 6.8 inches of run-off, was selected for flood control. In the initial development of the reservoir, this capacity is obtained with the spillway lip at Elevation 566.

(b) Conservation. - Based on analysis of stream flow and cost data for development of the reservoir at various elevations, a storage capacity of 69,000 acre-feet (10.1") below Elevation 573 was found to be the optimum capacity for conservation and power purposes at this site. The required storage of 46,000 acre-feet for flood control is then obtainable between Elevation 573 and Elevation 584. This plan was adopted for the ultimate development.

(c) Capacity Conditions Affecting Outlet and Spillway Design. - The principal factors affecting outlet and spillway design for this project were (1) desirability of sufficient outlet capacity to permit rapid emptying of the reservoir after floods, and (2) the necessity for outlet and spillway plans adaptable to a two-stage development.

(d) Computed Spillway Flood. - As defined in Engineer Bulletin, R. & H. No. 9, 1938, the "Computed Spillway Flood" is "the flood computed by means of the forecast worst storm, the highest run-off factors and the worst run-off producing combinations for the drainage area in question". For Blackwater Reservoir, the worst storm conditions were taken from the maximum precipitation values described in paragraph 3-03(c) with a total rainfall of 19.4 inches in 48 hours. Using a minimum average infiltration rate of 0.083 inch per hour, as determined in the hydrology report referred to in paragraph 3-01, the resulting run-off amounts to 15.8 inches. Including a base flow of 2 c.f.s. per square mile, the volume of the computed spillway flood is 110,600 acre-feet and the peak inflow, 49,300 second-feet.

3-06. Outlet Hydraulics. (a) Control Required. - The physical conditions at the Blackwater site are such that storage may be obtained over a wide range at a very low cost. Also, channel capacity immediately downstream is not a limiting factor because of the absence of possible flood damage. It is physically possible, therefore, to obtain a high degree of control economically.

(b) Determination of Reservoir Capacity. - The reservoir design flood was based on rainfall of about 100-year frequency plus a prior ground-saturating storm of moderate frequency. To arrive at such a storm, long-time rainfall records at Concord were studied. It was found from these records that a 100-year rainfall of 3 days' duration amounted to about 8 inches. With this storm a high run-off factor of 90 percent was applied. In order to justify this high run-off factor, a 3-day storm of 1-1/2 inches of rainfall per day was added prior to the 100-year 3-day storm. This gave a flood volume and peak that, in its effect on reservoir capacities, was about equivalent to the 1936 flood, the maximum flood of record. The flood obtained from these assumptions, however, must be considered as one of less frequent occurrence than once in 100 years because of the assumptions made of the 3-day prior rainfall and the high run-off coefficient. This flood, when routed through the reservoir with an outlet discharge of about 1,800 c.f.s., gave a spillway lip or maximum pool elevation of 566 feet. The storage capacity (46,000 acre-feet) at this elevation was adopted. As a check on the adequacy of this capacity, a design flood approaching the maximum probable flood was derived from the September 1932 storm, the greatest of record in the New England region. This check flood, as shown on Figure III-4, reaches an elevation of 567.1 in the reservoir, requires a maximum outlet discharge of 2000 second-feet and a moderate spillway discharge of 1200 second-feet. This is believed to be a satisfactory balance between storage and outlet capacity and represents a high degree of control of major floods.

(c) Determination of Outlet Capacity. - The dam will be built in two stages, initially for flood control only and ultimately raised to provide for flood control and power storage. Selection of the outlet capacities therefore required adjustment so that without permitting part-gate openings in operation the reservoir design outflow of 2000 c.f.s. during the peak of a flood would not be exceeded in either development. Since it will be practicable to operate the power plant of the final project during floods with a discharge of about 1000 c.f.s., it is necessary to provide outlets of such size that preferably but one outlet may be closed to compensate for this discharge and maintain the total reservoir outflow at approximately 2000 c.f.s. Excess gated outlet capacity was desirable in order to expedite emptying the reservoir after the flood. Also, it is desirable to keep the gate sizes the same. After consideration and correlation of the foregoing, the outlet capacity of 2000 c.f.s. is provided in the initial development with one 3'-6" x 6'-6" ungated outlet and one 3'-6" x 5'-3" gated outlet. Two additional 3'-6" x 5'-3" gated outlets are to be installed initially but not used for flood regulation. These additional outlets are available, however, for use in decreasing the time required to empty the reservoir after floods. In the ultimate development the ungated outlet will be closed off and no longer used. The design discharge of 2000 c.f.s. will then be attained at Elevation 584 by using two of the three gated openings. The third gated outlet is then available for additional capacity for emptying purposes. In the ultimate plan, the outflow can be held at approximately 2000 c.f.s. with the power plant in operation by using only one flood control outlet.

(d) Discharge of Freshets. - The storage capacity of the reservoir is comparatively large and with additional outlet capacity available for emptying after floods, the reservoir is entirely adequate for the discharge of freshets. It will rarely be filled to the spillway lip. The September 1938 flood routed through the reservoir showed a maximum reservoir stage below Elevation 555 and maximum storage of about 20,000 acre-feet.

(e) Time Required to Empty Reservoir. - With an assumed natural inflow of about 5 c.f.s. per square mile, the time required to empty the reservoir from full pool in the initial plan is about 9 days (see Figure III-4). For the ultimate plan the time required is 10 days. For the initial development the 1936 flood volume requires about 8 days to empty with an average inflow during this period of 1100 c.f.s. (see Figure III-5).

(f) Flood Discharge During Construction. - The contractor will be required to conduct his operations in such manner that the dam embankment and dikes shall be kept sufficiently in advance of the concreting operations in the spillway so that with a discharge of 10,000 c.f.s. over the spillway as constructed, the river will not overtop the dam embankment or the dikes. This discharge of 10,000 c.f.s. in addition to available outlet capacities will permit safe passage of a flood somewhat in excess of the maximum of record.

(g) Operation of Reservoir. - As outlined in paragraph 3-06(c), one ungated and three gated outlets are provided. In the initial plan the desired regulation will be obtained by using the ungated outlet and one gated outlet fully opened at all times. The two additional gated outlets will be kept closed unless it is desired to use them after floods for decreasing the time required to empty the reservoir. In the ultimate development, the ungated outlet will be closed off. The desired regulation is then obtained by operating two of the three gated outlets when required or by using one outlet with the power plant in operation. The third outlet is available as additional capacity for emptying the reservoir. Partial gate openings will not be required. The effect of Blackwater Reservoir on the maximum flood of record (March 1936) is shown on Figure III-5.

3-07. Spillway Hydraulics. (a) Criterion for Capacity. - The spillway is required to have sufficient capacity to pass the spillway design flood with a freeboard allowance of 5 feet for wave action and wind set-up under the following conditions: the reservoir full to the spillway lip at the start of the flood and all outlets blocked or inoperative throughout the flood period.

(b) Spillway Design Flood. - The computed spillway flood (see paragraph 3-05(d)) was routed through the reservoir to obtain the reservoir discharge and the maximum water surface elevation. The inflow ordinates of the computed spillway flood were then increased 50 and 100 percent and similarly routed through the reservoir. The data derived from these three routings are shown graphically on Figure III-6 for the initial plan and Figure III-8 for the ultimate development, where the percent of computed spillway flood is plotted (a) against the pool elevation in feet above mean sea level, and (b) against the peak inflows and outflows. After carefully considering all the factors itemized in paragraph 11 of Engineer Bulletin R. & H. No. 9, 1938, Subject: "Spillway Capacities", it was decided to adopt, as the "Spillway Design Flood" for the Blackwater

Dam, the computed spillway flood plus a factor of safety of 35 percent. Inflow, outflow and stage hydrographs for the spillway design flood under the initial plan are shown on Figure III-7. Corresponding data for the ultimate development are given on Figure III-9. General data concerning the flood are summarized as follows:

<u>Spillway Design Flood</u>

| | | |
|---|---|---|
| Percent of increase over Computed Spillway Flood | | 35% |
| Volume in acre-feet (including base flow) | | 148,300 |
| Equivalent volume in inches of run-off | | 22 |
| Peak inflow (c.f.s.) | | 66,500 |
| Myer Rating of peak inflow | | $5880\sqrt{DA}$ |
| Peak outflow (c.f.s.) | - initial stage | 42,800 |
| | - ultimate development | 40,500 |
| Maximum pool elevation | - initial stage | 579.0 |
| | - ultimate development | 595.0 |
| Freeboard provided | - initial stage | 5.0 |
| | - ultimate development | 5.0 |

# IV. HYDRAULIC DESIGN

4-01. Location of Discharge Structures. The spillway structure is located at the left end of the dam in the granite rock gorge forming the Blackwater River channel. The outlets are through the concrete spillway section. Discharge from the spillway and outlets is confined to this rock channel, entirely separate from the dam embankment.

4-02. Spillway Design. (a) Requirements.- The spillway is of concrete, with free overfall and is adequate to pass the spillway design flood with a 5-foot freeboard allowance. This requires a capacity of 42,800 c.f.s. in the initial development and 40,500 c.f.s. in the ultimate development. The spillway profile is designed to eliminate negative pressures on the crest and downstream face for a head equal to the surcharge at maximum discharge. The design of the spillway required that the lower portion of the spillway profile for the initial development must also satisfy the discharge conditions for the ultimate development when the spillway is raised to Elevation 584 M.S.L. The masonry line of both initial and ultimate developments was made to conform to that recommended by Creager and Justin (Hydro-Electric Handbook, Table 32), for a head of 13 feet initially and 11 feet ultimately. Adjustment of the required initial and ultimate spillway profiles in relation to each other was made by trial in order that the downstream spillway slope common to both would be satisfactory tangent to the upper portion of the spillway profiles. The selected masonry line was checked by: (1) the theoretical lower nappe as given by Creager and Justin's Hydro-Electric Handbook, and (2) the theoretical formula $x^2 = 1.8\ hy$ as used in the Technical Reports, Part 7, of the Miami Conservancy District. Both theoretical curves lie within the proposed masonry line for both initial and ultimate developments.

(b) Description of Spillway, and Approach and Discharge Channels.- For the initial development, the spillway has a length of 240 feet, crest at Elevation 566 M.S.L., and was designed for 13-foot maximum surcharge. For the ultimate development, the spillway has a length of 290 feet, crest at Elevation 584 M.S.L., and was designed for 11-foot maximum surcharge. A short bucket is provided at the base of the spillway to prevent direct impact of the falling sheet of water on the rock channel bed, the radius of the bucket being about one-fourth of the distance from Elevation 595 M.S.L. to the channel bed. No stilling basin or protection works was deemed necessary in this channel, as the rock is capable of withstanding high velocities without endangering the structure by erosion. In order to fit the 240-foot spillway of the initial development into the narrow gorge, the base of the spillway was stepped to conform to the channel, the left end of the spillway being provided with a minimum drop of 12 feet. The approach channel is the natural river channel, widened on the left bank with minimum approach depth of 5 feet below the spillway lip. The cross-section of the approach channel, taken at the upstream end of the penstock gatehouse, is about 6,800 square feet at Elevation 566 M.S.L. and 10,100 square feet at Elevation 579 M.S.L. This represents the minimum area of the approach channel. About 100 feet upstream from the dam the area is in excess of 12,000 square feet, at Elevation 579 M.S.L., giving a mean velocity of approach less than 3.6 feet per second for the spillway design flood. The discharge channel has ample capacity for all anticipated flood discharges. For the spillway design flood, the channel below the dam has ample capacity to prevent submergence. On the left bank, the channel has a minimum depth below the crest of the dam of 12 feet and a bed slope steep enough to carry the flow away at high velocities, thus preventing a hydraulic jump or submergence of this portion of the dam. However, to prevent possible overflow of the right bank with resulting flow near the toe of the dam embankment for a flood equal to the spillway design flood, the narrow rock gorge is to be enlarged to provide sufficient area at the most restricted section to give a mean velocity of about 22 feet per second. The flow conditions below the dam cannot be determined from computations. It appears that a hydraulic jump may form at the base of the highest portion of the spillway. In this event, the water surface would rise to about Elevation 538 M.S.L., with a mean velocity below the jump of but 7 feet per second. The floor over the spillway on either side, which accounts for about 3/4 of the total discharge will not have sufficient tail-water depths to form a hydraulic jump, therefore the major portion of the energy derived from the fall over the dam, with velocities varying from 25 feet per second to about 50 feet per second, will be retained in the flow. The channel area 80 feet below the dam is sufficient to provide for a mean velocity of about 15 feet per second at water-surface Elevation 538 M.S.L. It is desirable not to exceed this elevation because of anticipated wave action resulting from the mixing of flows with varying energy content and flow directions. Undoubtedly the energy content in the water due to velocity head is greater than that due to the average velocity of 15 feet per second. Assuming this energy head equal to about 6.0 feet (corresponding to a mean velocity of 20 feet per second) and deducting friction losses, the velocity at the smallest section (160 feet downstream) will be about 22 feet per second, the corresponding water-surface will be at Elevation 536 M.S.L. Below this control section the channel capacity increases.

(c) Spillway Discharge Capacity.- The spillway discharge capacity and corresponding pool elevation as affecting the dam was computed by means of the weir formula, $Q = CLH^{3/2}$, in which: $Q$ = the total discharge in c.f.s.; $L$ = the effective length of spillway weir; $H$ = total head in feet on the spillway crest, the difference between the elevation of the energy gradient at the weir and the crest of the weir. Because of the deep approach channel, frictional losses can be neglected. The maximum value of C used is 3.8, which is considered conservative for this type of weir. The rating curves for the spillway for both the initial and ultimate developments, and the coefficients used at the several intermediate heads below maximum, are shown in Figure IV-1.

4-03. Outlet Design. (a) The number of outlets was limited to the space available in the river gorge and the size, aside from capacity, to one that would readily pass debris. For these reasons, and the discharge capacities required, 4 outlets were found to be the optimum number.

(b) The size of outlets was determined by the gated outlet size required in the ultimate development to pass approximately 1000 second-feet each with reservoir at spillway crest. The cross-sectional area of such an outlet was computed to be approximately 18-1/2 square feet. It was considered desirable to have all the gates the same size and the height of the gate about twice its width, with no transitions or changes in area in the outlets other than the bellmouth entrance. The selected dimensions of the outlet and gate passage are 5 ft. 3 in. high and 3 ft. 6 in. wide. Three such gated outlets are provided.

(c) The capacity of this size of outlet was then determined with reservoir Elevation 566 M.S.L. the spillway crest elevation of the initial development, and found to be 870 c.f.s. Since it is proposed to limit the outlet design discharge in the initial development to 2000 second-feet, without regulation by partial gate openings, it was found advantageous to provide an ungated outlet to discharge approximately 1130 second-feet. This, with one gated outlet left open at all times, gives the required flood control outlet capacity, no gate operation being necessary except after a major flood when the remaining two gated outlets will be opened to permit rapid emptying of the reservoir. Similarly, in the ultimate development, during a flood exceeding the power storage at Elevation 573 M.S.L., two of the outlets will be opened as required.

(d) Description of Outlets.- (1) The approach channel for the outlets is located in the deepest part of the rock gorge forming the river channel. The channel has ample capacity for all flows permitting low velocities of approach. The invert of the 3 gated outlets is about 5 feet above the natural rock bed elevation; the transition from the natural channel to the intake elevation is a flat curve of about 50-foot radius. The single ungated outlet is located at the left side of the channel and requires excavating the approach channel for about 75 feet upstream. The channel bed is to be excavated to Elevation 513 M.S.L.. 2 feet below the outlet invert.

(2) Trash bars are provided for the 4 outlets. They are semi-elliptical in section, their long axis set parallel to the flow to minimize flow disturbance at the entrance. Mean velocities through the trash bars will not exceed about 12 feet per second.

(3) The transition of the gated outlets, from entrance section at the upstream end of the trash bars to the gate passage, is a bellmouth, about 17 feet long, laid out on easy curves extending to within 5 feet of the emergency gates. The top of the transition is shaped to a quarter ellipse, with the horizontal axis about twice the vertical axis. Mean velocities for this intake are given in Figure IV-2 for reservoir level at Elevation 584 and 566, M.S.L. The bellmouth for the ungated outlets is similar to that for the gated outlets, the transition from the entrance area of 145

square feet (10-3/4 ft. x 13-1/2 ft.) to the 3-1/2 ft. x 6-1/2 ft. passageway being made in a distance of 15 feet.

(4) The gated outlets are provided with air vents in the roof immediately downstream from both the emergency and service gates. It was considered advisable to vent the emergency gate as well as the service gate to permit its use as a spare gate, although its vent was not carried above maximum tailwater but well above that anticipated with all 4 outlets discharging. Vent capacity provided was made to conform to the data in Figure 2 given on page 180 of "Dams and Control Works", U. S. Bureau of Reclamation (second edition).

(5) The gate guide slots produce considerable disturbance to flow, necessitating lining the gate passageways in the vicinity of these slots.

(6) The downstream end of all outlets was curved downward, the inverts meeting the bucket of the spillway on a parabolic curve that is flatter than the outflow trajectory with maximum pool elevation. The outlets downstream from the bellmouth intake are uniform in cross-section and area throughout their length.

(7) No stilling basin is provided as the outflow is into a granite rock channel. Consideration was given to the use of a splitter at the outlet portal similar to that devised for the Pittsburgh District, but these were not considered necessary from the standpoint of channel erosion downstream. Their use could be justified to prevent ride-up along the right bank of the high velocity jets from the outlets. To prevent this, it is proposed to carry the channel at Elevation 512 M.S.L. for a distance of about 100 feet downstream, in line with the outlet nearest the right bank. At this point the channel bank makes an angle of about 20 degrees with the line of flow and would not cause excessive ride-up; also, since there will not be sufficient tailwater depth to drown out the outlet discharge, there will be considerable spreading of the flow along the channel floor, with resulting dissipation of energy and velocity.

(e) Computation of Discharge Capacity.- For outlets flowing full, the discharge capacity was determined on the basis of the outlets discharging as an orifice, i.e., $Q = A \sqrt{2gh}$, in which Q = the total discharge in c.f.s., A = the cross-sectional area of the outlet in square feet, and h = the net head in feet. The total head was taken as the head between pond elevation and a point just below the crown of the outlet portal. The net head, h, was determined by deducting from the gross head all head losses in the outlet, including velocity head at the portal. Computations for head losses are given in Figures IV-3 to IV-6. Hydraulic characteristics of the outlets are shown in Figure IV-7. For outlets flowing part full, pond elevations were computed for several discharges, considering the control near the downstream end at the point where the outlet invert is at critical slope for the given discharge. The rating curves for the ungated and gated outlets are shown in Figure IV-8. Rating curves shown in Figure IV-9 are for the outlets operated in the initial development as outlined in paragraph 3-06(g); one curve for the ungated and one gated outlet remaining open at all times, the other curve for all outlets discharging after the flood, if required to empty the reservoir rapidly. Figure IV-10 shows the rating curves for one, two, and three gated outlets of the ultimate development, wherein one outlet is used for flood control as required in combination with the power plant discharge, two outlets are used (with no power plant discharge) as required for discharge during floods causing the power pool elevation to exceed 573 M.S.L., and all three outlets if required to empty the reservoir after the flood. Figure IV-11 shows the computations for determining the hydraulic and energy gradients in the gated outlets for pool Elevation 584 M.S.L., and Figure IV-12 shows the hydraulic and energy gradients in the gated outlets with reservoir at spillway crest for both the initial and final developments. There is no tailwater rating curve available for the outlets. However, studies made on the discharge channel capacity indicated that for maximum outlet discharge the tailwater could not rise high enough to affect the discharge capacities of the outlets.

4-04. Freeboard. (a) General. - Sufficient freeboard is provided against overtopping the dam by wave action at the maximum water surface. Because of the unusually sheltered location of the Blackwater Dam with respect to the maximum possible conditions producing wave action and wind set up, the minimum freeboard of 5 feet was provided. This value of freeboard has been checked in accordance with the general rules set forth in Engineer Bulletin R. & H. No. 9, 1938, and found adequate.

(b) Allowance for Wind Set-up. - Using the formula of the Lorentz Zuiderzee Commission of Holland,

$$S = .00125 \frac{V^2F}{D} \cos A$$

in which S = wind set-up in feet, V = velocity of wind in miles per hour (taken as 80 miles per hour for this vicinity), F = fetch in miles (about 5-1/2 miles for a maximum value), D = depth of reservoir in feet (about 40 feet with reservoir Elevation 579) and A = angle of wind and fetch (taken as zero for most extreme conditions), the wind set-up is computed as 0.2 foot per mile of fetch or 1.1 feet for a fetch of 5-1/2 miles.

(c) Allowance for wave action was computed by the Stevenson formula as modified by Molitor (see page 984, Trans. Am. Soc. C.E., Vol. 100, 1935)

$$h = 0.17 \sqrt{VF} + 2.5 - \sqrt[4]{F}$$

in which h = wave height in feet and V and F are defined and evaluated as in the preceding paragraph.

(d) Allowance for ride-up of waves was taken as 1.4 times the total height of wave as computed in the foregoing paragraph.

(e) Total Freeboard Required. - Two possibilities were tried for the dam. That producing the maximum condition was for the maximum reservoir fetch of 5.5 miles for wind set-up, assuming that the maximum water surface produced by this set up is transmitted to the dam. Since it appears that the wind waves will be dissipated in passing perpendicular to their path of travel to the pool above the dam, the maximum fetch applicable above the dam is about 1/2 mile. The computed value of wind set-up and wave height produced under these conditions is less than 5 feet. For the dikes a fetch of 1-1/2 miles was used and the total computed value of wind set-up and wave height found to be less than 5 feet.

4-05. Model Studies. No hydraulic model tests have been made. Such tests would be useful only for checking the adequacy of the channel below the spillway to prevent overtopping the right bank (see paragraph 4-02(b)).

4-06. Conclusions. The foregoing analysis of the hydraulic design shows that the structures provided satisfy the following criteria:

(1) The spillway and outlet capacity is so great that there is no danger of overtopping.

(2) The freeboard is so great that there is no danger of overtopping by waves.

(3) There is no opportunity for the free passage of water from the upstream to the downstream face.

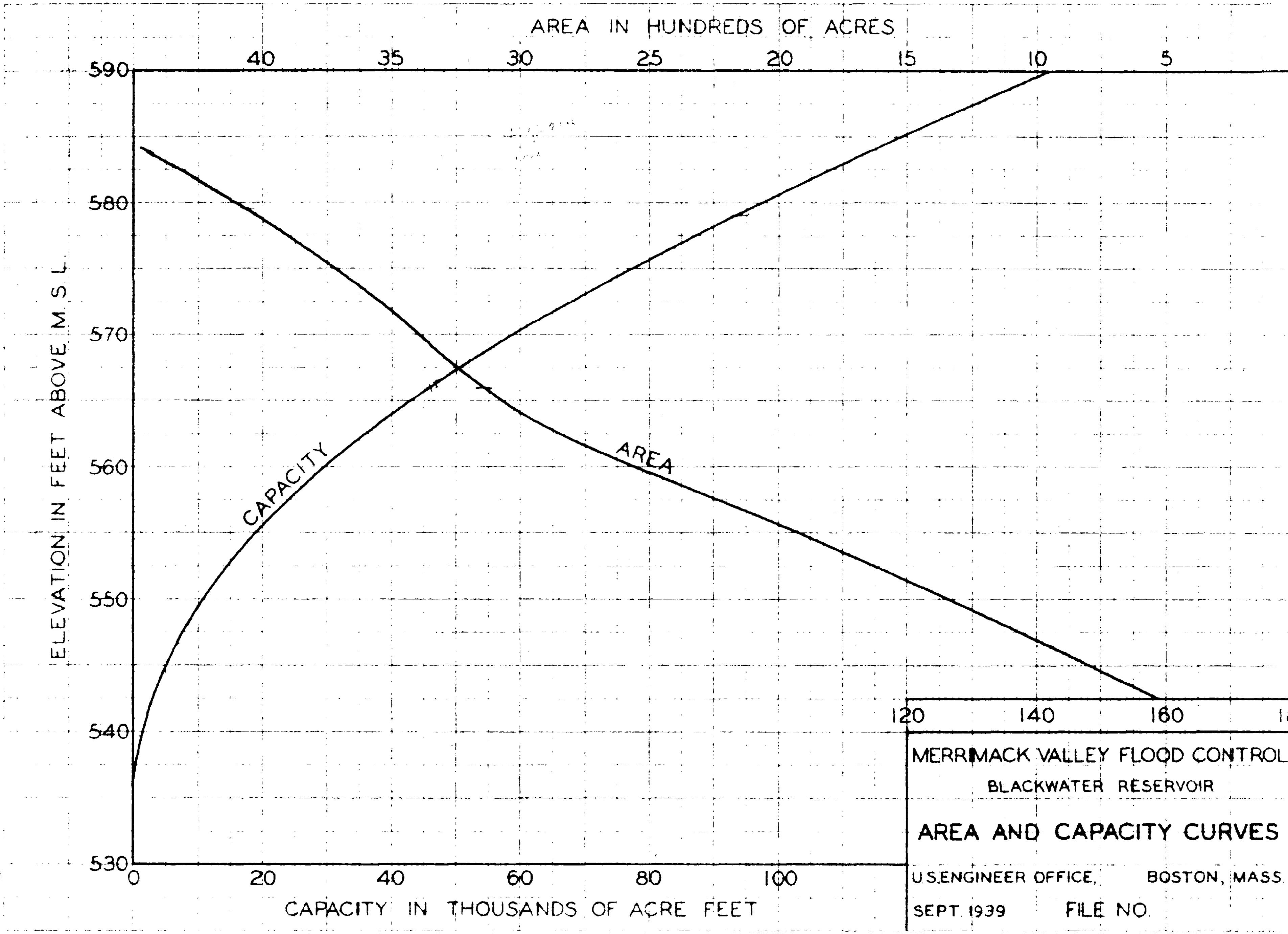
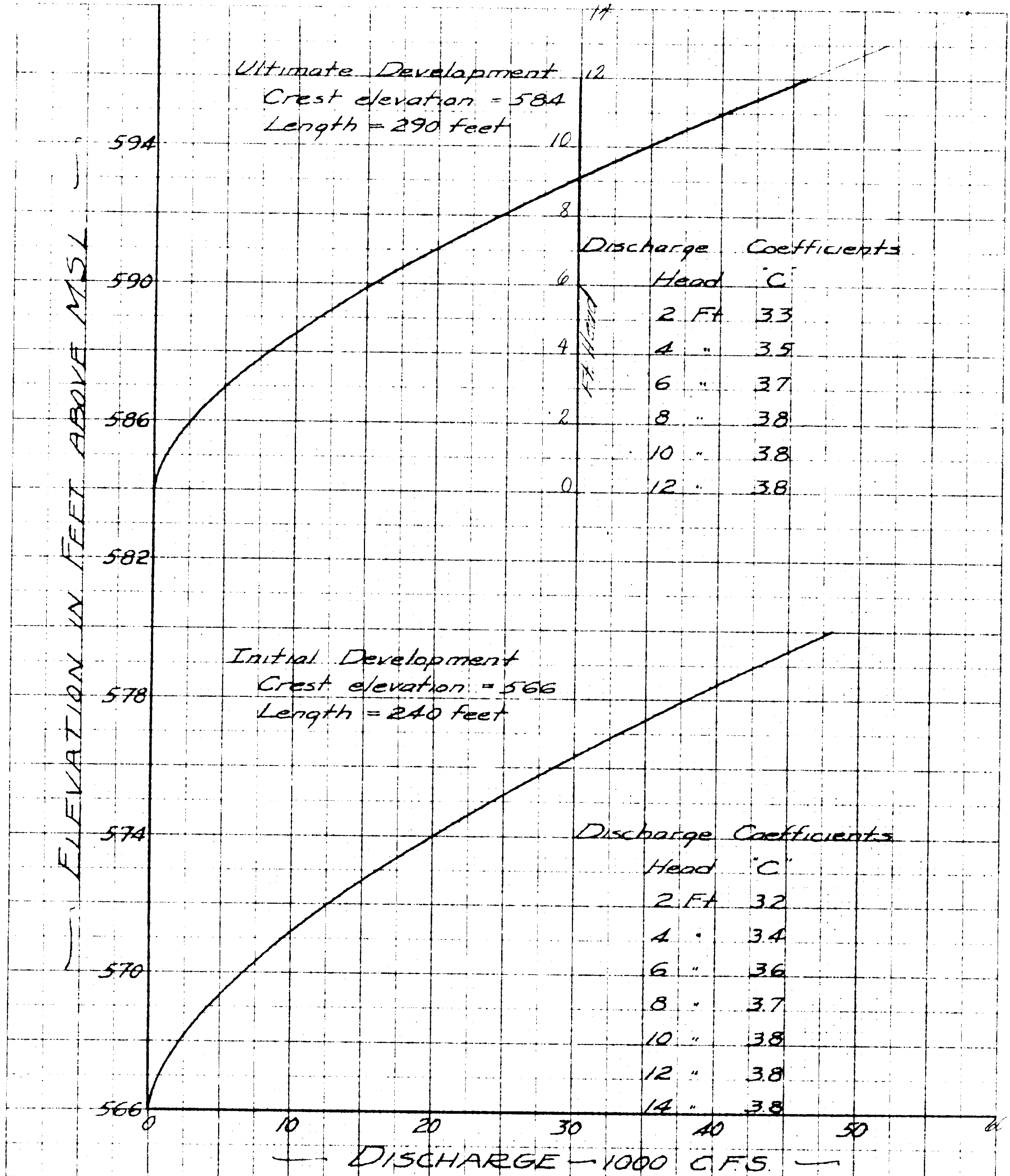

AREA IN HUNDREDS OF ACRES
ELEVATION IN FEET ABOVE M.S.L.
CAPACITY IN THOUSANDS OF ACRE FEET
CAPACITY
AREA
MERRIMACK VALLEY FLOOD CONTROL
BLACKWATER RESERVOIR
AREA AND CAPACITY CURVES
U.S. ENGINEER OFFICE, BOSTON, MASS.
SEPT. 1939 FILE NO.

FIG III-3




MERRIMACK VALLEY FLOOD CONTROL

BLACKWATER RESERVOIR

SPILLWAY DISCHARGE

RATING CURVES

U.S. ENGINEER OFFICE BOSTON, MASS

JANUARY 1940

FIG IV - 1

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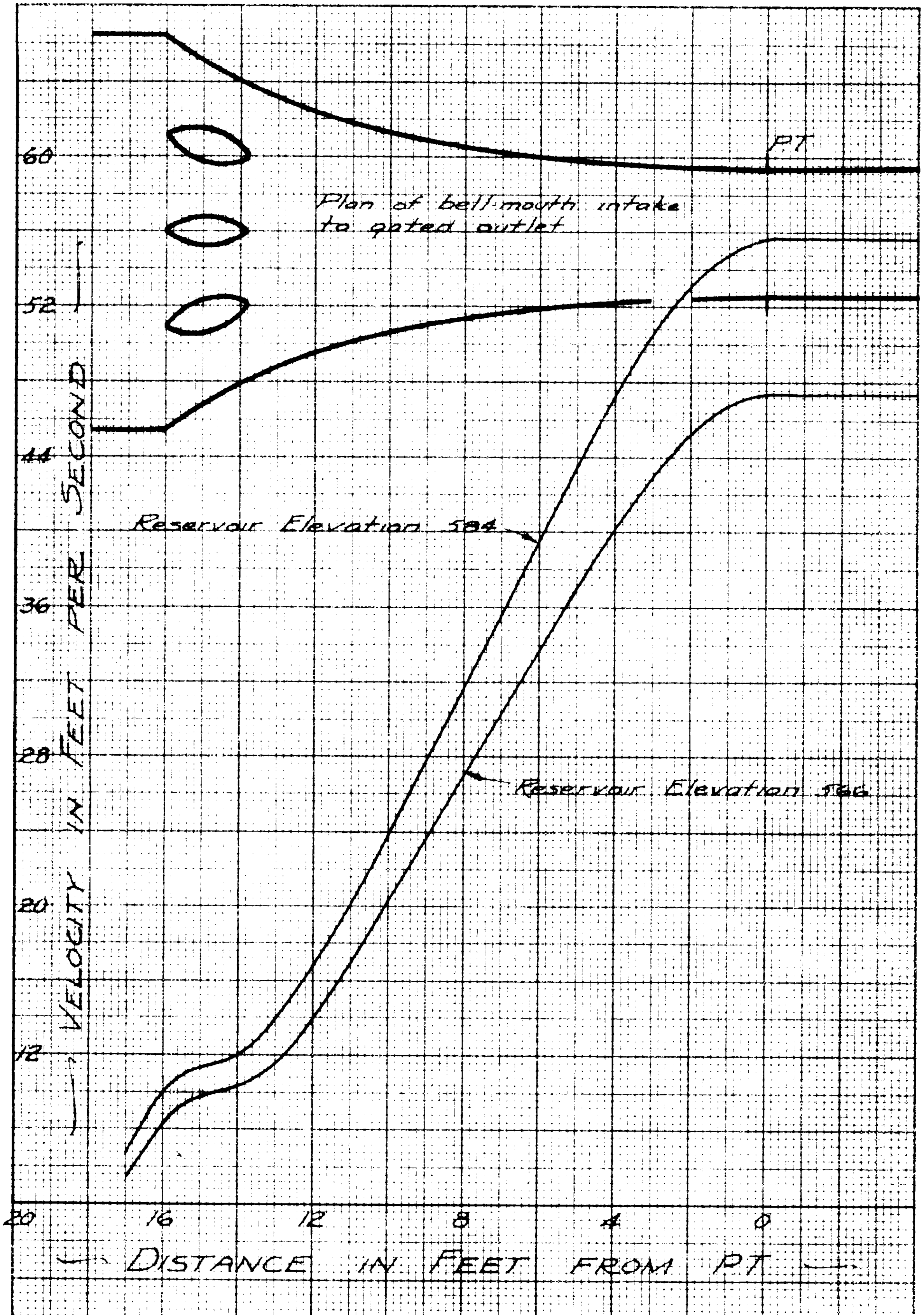



MERRIMACK VALLEY FLOOD CONTROL
BLACKWATER RESERVOIR
INTAKE VELOCITIES
GATED OUTLETS
U.S. ENGINEER OFFICE BOSTON MASS
JANUARY 1940

FIG IV-2

Subject Blackwater Dam

Computation Discharge Capacity of Gated Outlets

Computed by K. J. Checked by Ec Date 12/15/39

Outlet Dimensions

Bell-mouth intake — length 17 feet

— area at entrance 158 sq. ft.

Gate passage — length 47 ft.

— height 5.25 ft.

— width 3.50 "

— invert elev. 515 "

Discharge required per outlet for ultimate development (pool elev 584 ft.) = 1000 cfs

Hydraulic Characteristics

Area — $A = 5.25 \times 3.50$ = 18.4 sq. ft.

Wetted perimeter = $2(5.25 + 3.50)$ = 17.5 ft.

Hydraulic radius — $R = 18.4 \div 17.5$ = 1.05 "

Assumed value of "n" (Kutters) = 0.013

Chezy coefficient "C" = 118

Head Losses

Let $v$ = velocity in gate section

1. $h_1$ = Entrance loss in bell-mouth (including trash bars) $= 0.05 \dfrac{v^2}{2g}$

2. $h_2$ = Friction losses $= \dfrac{64.4\,L}{C^2 R} \cdot \dfrac{v^2}{2g}$

where $L = 54$ (47 ft. of section 5.25 × 3.5' and 7 ft. of bell-mouth section upstream from point of tangency with gate passage.)

$C = 118$

$R = 1.05$

$$h_2 = \frac{64.4 \times 54}{118^2 \times 1.05} \cdot \frac{v^2}{2g} = \qquad 0.24\,\frac{v^2}{2g}$$

3. $h_3$ = Eddy losses from gate slots = 0.06 "

4. $h_4$ = Velocity head at outlet portal = 1.00 "
(Assuming no recovery of head)

FIG. IV - 3

WAR DEPARTMENT

U. S. ENGINEER OFFICE, BOSTON, MASS.

Page

Subject Blackwater Dam

Computation Discharge Capacity of Gated Outlets (con)

Computed by H. J. Checked by Ec Date 12/15/39

Total head, $H = \Sigma(h_1, h_2, h_3, h_4) = 1.35 \frac{v^2}{2g}$

Discharge per outlet -

$$v = \sqrt{\frac{2gh}{1.35}} = 6.91\sqrt{H}$$

$$Q = Av = 18.4 \times 6.91\sqrt{H} = 127\sqrt{H}$$

Datum for head for outlets flowing full taken as elevation 519, slightly below top of outlet portal.

Total head to spillway crest -

Initial development $= 566 - 519 = 47$ ft.

Ultimate " $= 584 - 519 = 65$ "

Outlet discharge with water surface at spillway crest.

Initial development

$$Q = 127\sqrt{47} = 870 \text{ cfs}$$

$$v = 6.9\sqrt{47} = 47 \text{ ft/sec.}$$

Ultimate development

$$Q = 127\sqrt{65} = 1020 \text{ cfs}$$

$$v = 6.9\sqrt{65} = 56 \text{ ft/sec.}$$

Subject Blackwater Dam
Computation Discharge Capacity of Ungated Outlet
Computed by K. J. Checked by E.C. Date 12/5/39

Outlet dimensions

| | | |
|---|---|---|
| Bell-mouth intake | – length | 15 ft. |
| | area at entrance | 145 sq ft. |
| Conduit | – length | 41 ft. |
| | height | 6.5 " |
| | width | 3.5 " |

Discharge capacity required about 1130 cfs with reservoir at elevation 566. This discharge combined with discharge of 870 cfs from one gated outlet equals outlet design discharge of 2000 c.f.s. The ungated outlet will be used only for the initial development, and permanently closed in the ultimate development.

Hydraulic Characteristics

| | |
|---|---|
| Area – A = 6.50 × 3.50 = | 22.8 sq. ft. |
| Wetted perimeter = 2(6.5+3.5) = | 20.0 ft. |
| Hydraulic radius – R = 22.8 ÷ 20.0 = | 1.14 " |
| Assumed value of "n" (Kutters) = | 0.013 |
| Chezy coefficient "C" = | 119 |

Head Losses

Let $v$ = velocity in conduit section.

1. $h_1$ = Entrance loss in bell-mouth (Including trash bars) = $0.05 \frac{v^2}{2g}$

2. $h_2$ = Friction losses = $\frac{64.4\,L}{C^2 R} \cdot \frac{v^2}{2g}$

$L = 46$ (41 ft of section 6.5' × 3.5' and 5 ft of bell-mouth section upstream from point of tangency)

$C = 119$

$R = 1.14$

$$h_2 = \frac{64.4 \times 46}{119^2 \times 1.14} \times \frac{v^2}{2g} = 0.18 \frac{v^2}{2g}$$

FIG. IV-5

Subject Blackwater Dam
Computation Discharge Capacity of Ungated Outlet (con)
Computed by M. J. Checked by E Date 12/15/39

3. $h_3$ = Velocity head at outlet portal $= 1.00 \frac{v^2}{2g}$
(Assuming no recovery of head)

Total Head $H = \Sigma(h_1 h_2 h_3) = \quad 1.23 \frac{v^2}{2g}$

Discharge –

$$v = \sqrt{\frac{2gh}{1.23}} = 7.24\sqrt{H}$$

$$Q = vA = 22.8 \times 7.24\sqrt{H} = 165\sqrt{H}$$

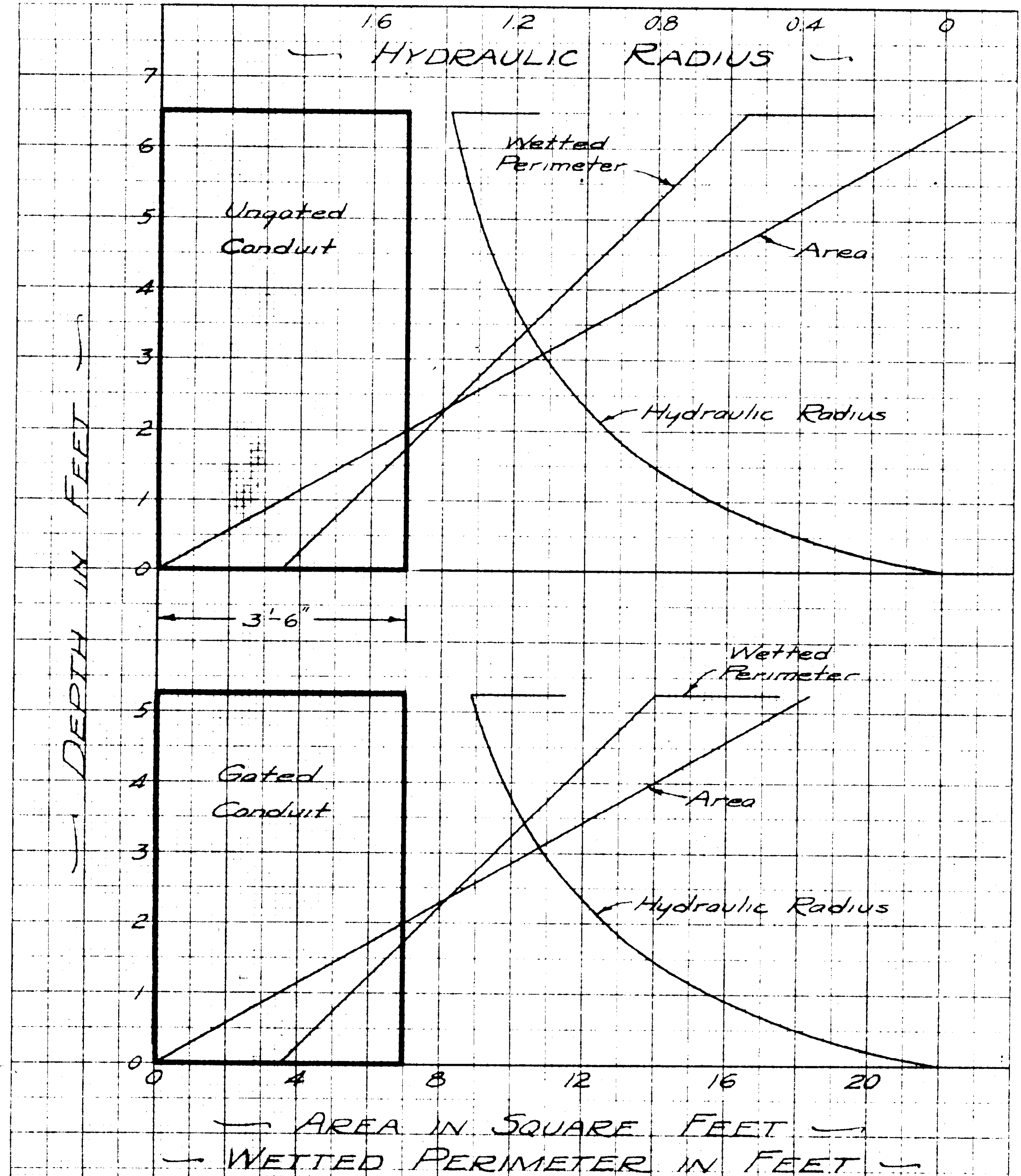
Datum for head for outlets flowing full) taken as elevation 520, slightly below top of outlet portal

Total head to spillway crest
$= 566 - 520 = 46$ ft.

Outlet discharge with water surface at spillway crest

$$v = 7.25\sqrt{46} = 49 \text{ ft/sec.}$$

$$Q = 165\sqrt{46} = 1120 \text{ cfs}$$

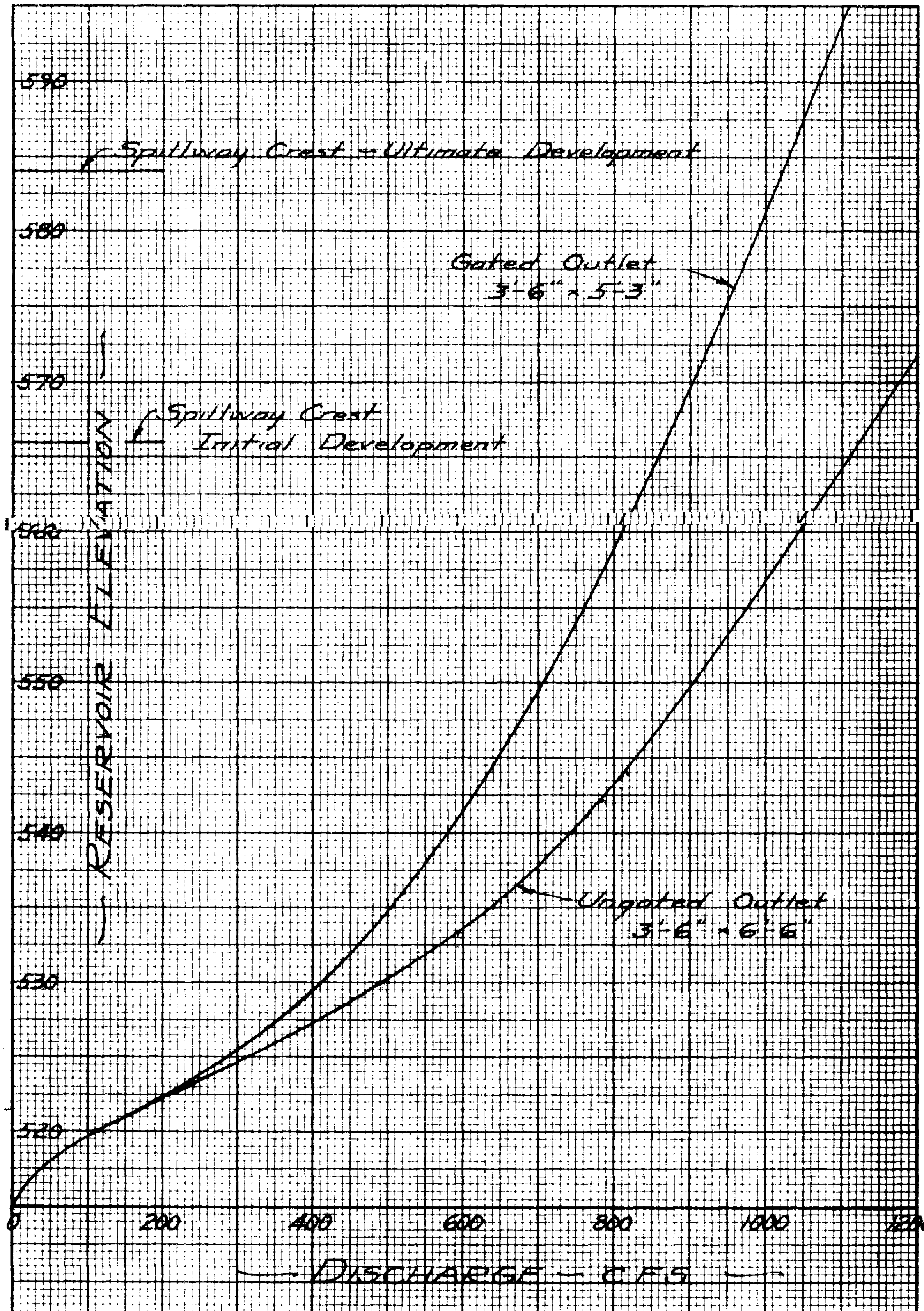



MERRIMACK VALLEY FLOOD CONTROL
BLACKWATER RESERVOIR
HYDRAULIC CHARACTERISTICS
OF OUTLETS

U.S. ENGINEER OFFICE BOSTON, MASS
JANUARY 1940

FIG IV-7

WAR DEPARTMENT CORPS OF ENGINEERS U. S. ARMY




MERRIMACK VALLEY FLOOD CONTROL

BLACKWATER RESERVOIR

RATING CURVES OF OUTLETS

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JANUARY 1940

FIG. IV 8

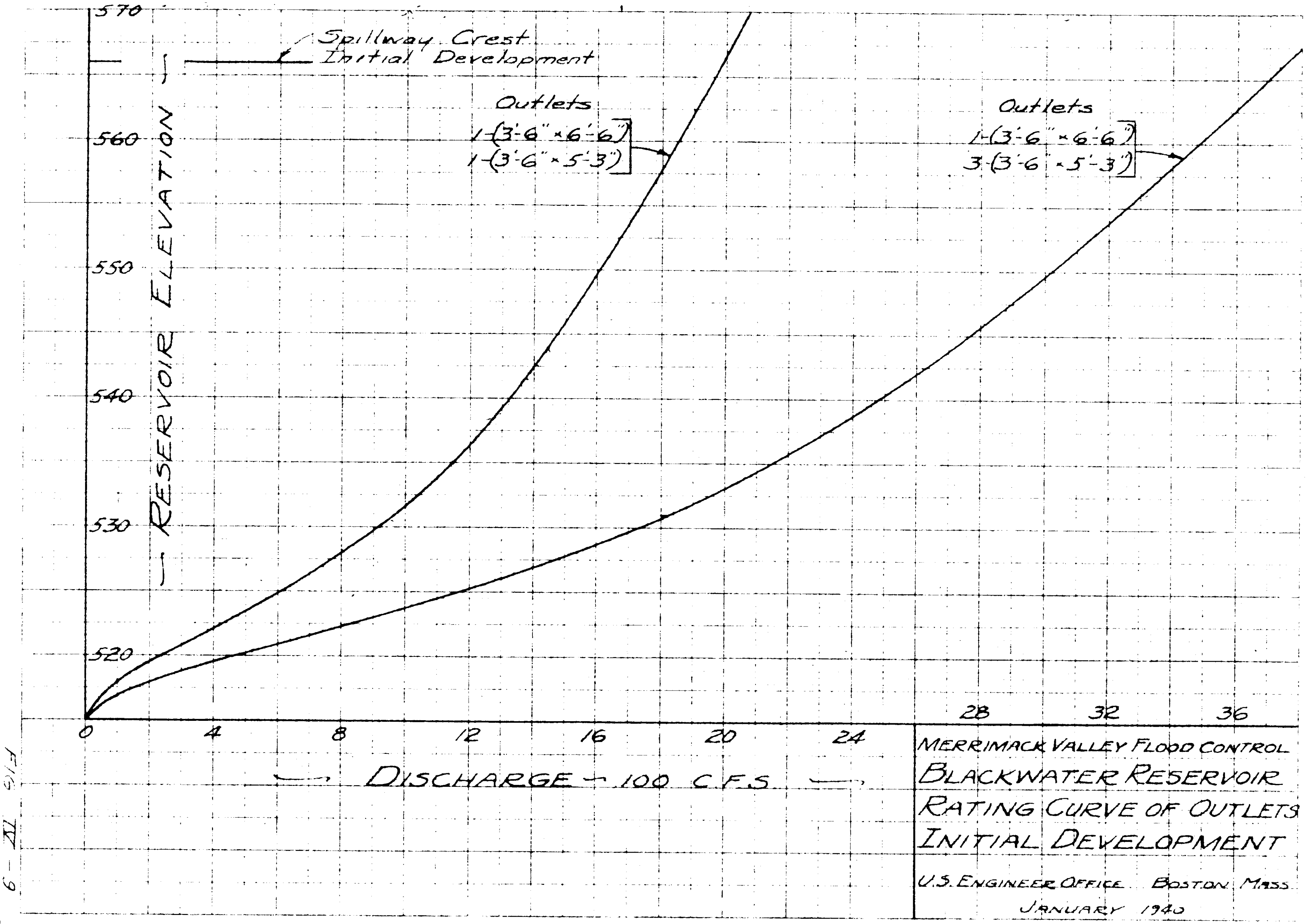

570
560
550
540
530
520
0
4
8
12
16
20
24
28
32
36
Spillway Crest
Initial Development
Reservoir Elevation
Outlets
1-(3'-6" × 6'-6")
1-(3'-6" × 5'-3")
Outlets
1-(3'-6" × 6'-6")
3-(3'-6" × 5'-3")
Discharge – 100 C.F.S.
Merrimack Valley Flood Control
Blackwater Reservoir
Rating Curve of Outlets
Initial Development
U.S. Engineer Office Boston, Mass.
January 1940

FIG IV - 9

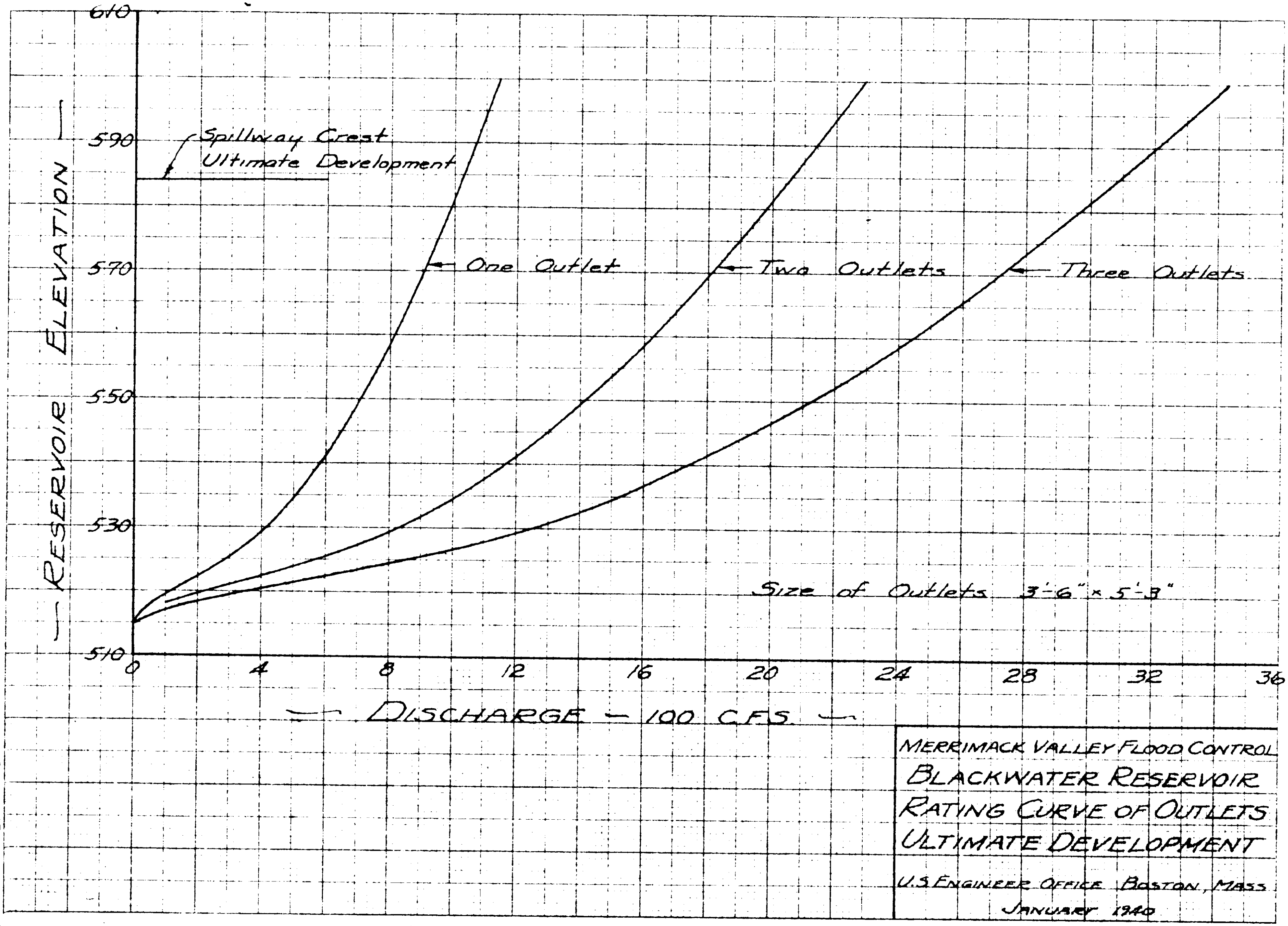

610
590
570
550
530
510
Reservoir Elevation
Spillway Crest
Ultimate Development
One Outlet
Two Outlets
Three Outlets
Size of Outlets 3'-6" × 5'-3"
0
4
8
12
16
20
24
28
32
36
Discharge — 100 C.F.S.
Merrimack Valley Flood Control
Blackwater Reservoir
Rating Curve of Outlets
Ultimate Development
U.S. Engineer Office Boston, Mass
January 1940

FIG IV-10

WAR DEPARTMENT

U. S. ENGINEER OFFICE, BOSTON, MASS.

Page

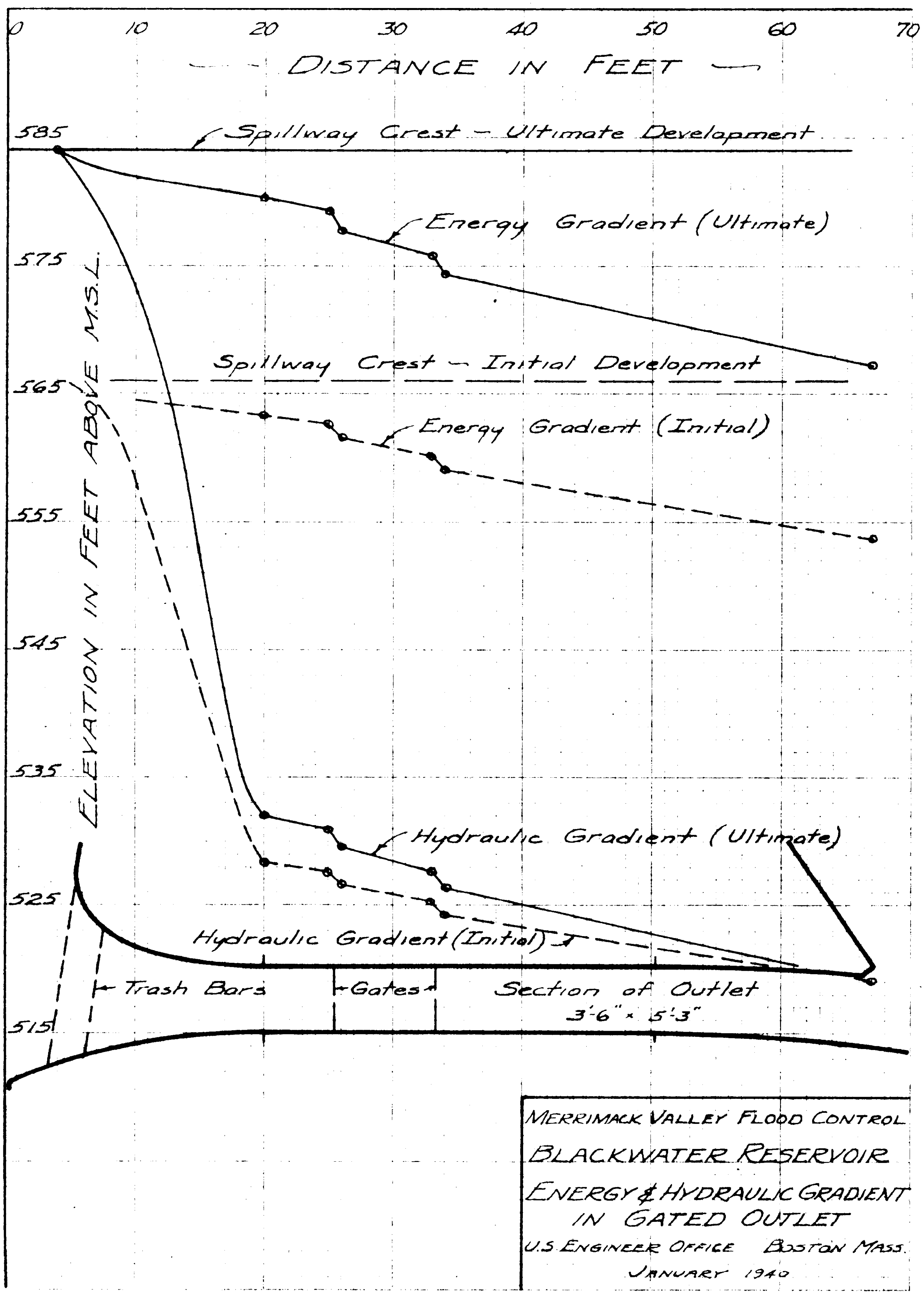
Subject: Blackwater Reservoir - Gated Outlets

Computation: Outlets - Energy & Hydraulic Gradient - Ultimate

Computed by: E.C. Checked by: K. J. Date: 1/9/40

| Station | Description of Head Loss | Head Loss in terms of velocity head | Head Loss in feet | Elev. of Energy Gradient | Elev. of Hydraulic Gradient | Velocity Head |
|---|---|---|---|---|---|---|
| 0+04 | | | | 584.00 | 584.0 | $\frac{v^2}{2g} = 0$ ($v = 0$) |
| | Entrance loss | $0.05 \frac{v^2}{2g}$ | 2.41 | | | |
| | Friction loss $\frac{7}{54} \times 0.24 =$ | 0.03 " | 1.44 | | | |
| 0+20 | | | | 580.15 | 532.00 | $\frac{v^2}{2g} = 48.15$ ($v = 55.65$) |
| | Friction loss $\frac{5}{54} \times 0.24 =$ | 0.02 " | 0.96 | | | |
| 0+25 | | | | 579.19 | 531.04 | |
| | Gate slot | 0.03 " | 1.44 | | | |
| 0+26 | | | | 577.75 | 529.60 | |
| | Friction loss (8 ft.) $\frac{8}{54} \times 0.24 =$ | 0.04 " | 1.93 | | | |
| 0+33 | | | | 575.82 | 527.67 | |
| | Gate slot | 0.03 " | 1.44 | | | |
| 0+34 | | | | 574.38 | 526.23 | |
| | Friction loss (34 ft.) $\frac{34}{54} \times 0.24 =$ | 0.15 " | 7.22 | | | |
| 0+67 | | | | 567.16 | 519.01 | |
| | Velocity head (Assuming no recovery) | 1.00 " | 48.15 | | | |
| | | | | 519.01 | | |

FIG. IV-11



0 10 20 30 40 50 60 70
DISTANCE IN FEET
585
575
565
555
545
535
525
515
ELEVATION IN FEET ABOVE M.S.L.
Spillway Crest - Ultimate Development
Energy Gradient (Ultimate)
Spillway Crest - Initial Development
Energy Gradient (Initial)
Hydraulic Gradient (Ultimate)
Hydraulic Gradient (Initial)
Trash Bars
Gates
Section of Outlet
3'-6" × 5'-3"
MERRIMACK VALLEY FLOOD CONTROL
BLACKWATER RESERVOIR
ENERGY & HYDRAULIC GRADIENT
IN GATED OUTLET
U.S. ENGINEER OFFICE BOSTON MASS.
JANUARY 1940


FIG. IV - 12